November 14, 2011

# FOUNDATION INVESTIGATION AND DESIGN REPORT

## HIGHWAY 11 MATTAWISHKWIA RIVER BRIDGE REPLACEMENT
## SITE 39W-33, HEARST, ONTARIO
## MINISTRY OF TRANSPORTATION, ONTARIO, GWP 164-98-00

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REPORT

Golder Associates

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## FOUNDATION INVESTIGATION REPORT

## HIGHWAY 11 MATTAWISHKWIA RIVER BRIDGE REPLACEMENT

## SITE 39W-33, HEARST, ONTARIO

## MINISTRY OF TRANSPORTATION, ONTARIO, GWP 164-98-00

# 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Ltd. (MMM) on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the Mattawishkwia River Bridge replacement on Highway 11 on the east side of Hearst, Ontario. The general location of this section of Highway 11 alignment is shown on the Key Plan on Drawing 1.

The terms of reference and scope of work for the foundation investigation and design are outlined in MTO's Request for Proposal dated July 16, 2010. Golder's proposal (P0-1191-0038, dated August 9, 2010) for foundation engineering services associated with the bridge replacement project is contained in Section 6.8 of MMM's Technical Proposal that forms part of the Consultant's Agreement (Purchase Order Number 5008-E-0077) for this project. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated November 16, 2010. The General Arrangement (GA) drawing for the proposed replacement structure was provided to Golder by MMM.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement structure by borehole drilling, rock coring, in situ testing and laboratory testing on selected samples.

The investigation was supplemented with information contained in reports by MTC (1978)[1] and PML (2008)[2] from the MTO GEOCRES database.

# 2.0 SITE DESCRIPTION

The site is located along Highway 11 and crosses the Mattawishkwia River in the Town of Hearst, Ontario. The surrounding land is mainly residential with Ontario Northland Railway (ONR) tracks running parallel and about 20 m south of Highway 11. The riverbanks are generally grass covered with trees and small shrubs located beyond the edges of the riverbanks. Cobbles and boulders are visible within the creek bed. Based on the GA provided by MMM, the river is about 50 m wide and about 5 m in depth and, at the time of our field investigation (March 2011), the water was generally about 0.6 m deep (below ice surface). During Golder's foundation investigation carried out in March 2011, the ice/water level was generally measured at Elevation 231.2 m. The high water level (50 year) is reported to be Elevation 234.7 m.

We understand that the existing Mattawishkwia River Bridge was constructed in 1938 and has an overall deck length of about 72 m and width of about 10 m. The top of both the west and east embankments are at about Elevation 237.3 m and are about 6.8 m high relative to the river bed.

From the preliminary report by PML in 2008, the existing bridge is a continuous, four-span reinforced concrete 'T' beam superstructure with an asphalt wearing surface. The three piers are supported by spread footings with both end spans cantilevered such that there are no abutments. According to PML's preliminary report, the east and west piers have tilted inward toward the river and the centre pier does not appear to have moved/tilted. MTC's 1978 foundation investigation report indicates that the tilting of the piers was first observed in 1956 and

[1] Foundation Investigation Report for Mattawishkwia River Bridge, W.P. 236-77-00, Site 39W-33, Hwy 11, District 16, Cochrane, GEOCRES No. 42G-16, dated November, 1978, by the Ministry of Transportation and Communications (MTC), Highway Engineering Division, Engineering Materials Office – Soil Mechanics Section

[2] Preliminary Foundation Investigation and Design Report for Replacement of Mattawishkwia River Bridge, Highway 11, Site No. 39W-033, GWP 154-98-00, Town of Hearst, District of New Liskeard, GEOCRES No. 42G-29, dated July 3, 2009, by Peto MacCallum Ltd. (PML)

settlement and undermining of the approach slabs was also observed at that time. In 1967, the bridge was rehabilitated by grouting the east pier, placing gabions on the east front slope and lengthening the approach slabs from about 3 m to 6 m. The tilting continued in spite of the rehabilitation and, in 1982, the gabion baskets were removed; a sheet-pile tie-back anchor wall was installed at both ends of the bridge to allow for the removal of the gabion baskets and to retain the front embankment slope and lower the grade in front of the retaining wall next to the river. We understand that the sheet-piles were driven to Elevation 230.7 m and 229.6 m on the west and east side of the bridge, respectively. Deadman anchors were installed at both ends about 18 m back from the sheet-pile wall.

# 3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation associated with the proposed bridge replacement was carried out between March 16 and 24, 2011, during which time a total of six (6) boreholes (MA-1 to MA-6) were advanced for the proposed bridge replacement, as shown on Drawing 1. Borehole MA-1 was advanced using a track-mounted D-50 and Boreholes MA-2 to MA-6 were advanced using a skid-mounted D-25. The drilling equipment was supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The D-25 was mounted on a raft in order to advance Boreholes MA-2 to MA-5 from the ice surface. The Record of Boreholes and Drillhole sheets are provided in Appendix A.

The current investigation was supplemented by boreholes advanced by others and in particular by boreholes as follows:

- Two (2) boreholes identified as Boreholes 5 and 4 were advanced by MTC in August 1978 to auger refusal at depths of 10.1 m and 8.8 m, respectively, in the vicinity of the proposed west and east abutments, respectively; and
- Two (2) boreholes identified as Boreholes 102 and 103 were advanced by PML in November 2008 to depths of 9.2 m and 10.2 m, respectively, within 20 m of the west and east approach, respectively.

The locations of the supplemental boreholes in the vicinity of the bridge are shown on Drawing 1 and the Record of Boreholes are provided in Appendix B.

The boreholes for the current investigation were advanced using NW casing with wash boring and NQ coring, except at Borehole MA-1 where H-size casing and coring was used (HW and HQ). Where possible, soil samples were generally obtained continuously or at intervals of depth of about 0.75 m to 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) with cobbles and boulders and/or bedrock generally recovered using the core barrel. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).

The boreholes during the current investigation were advanced to depths ranging between 5.1 m and 11.5 m below existing ground or water surface.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendix A. A piezometer was installed in Borehole MA-6 to permit monitoring of the groundwater level at this location. The piezometer consists of 50 mm diameter PVC pipe, with a 1.5 m long slotted screen installed within the silty sand to gravelly sand deposit. The piezometer details and water level readings are described on the Record of Borehole sheet in Appendix A.

The fieldwork was supervised throughout by a member of our technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. The results of the laboratory testing are included in Appendix C.

The as-drilled borehole locations for the current investigation were measured in the field relative to stations marked by MMM on the existing pavement as well as relative to the existing bridge. The as-drilled borehole location for MA-1 was surveyed by MMM and the MTM NAD 83 coordinates and ground surface elevation were forwarded to Golder. Further, for Boreholes MA-2 to MA-6, Golder marked the existing bridge for each borehole and referenced as-drilled locations and elevations to the paint markings to which MMM surveyed and provided coordinates and elevations for each of the markings. The ground or ice surface elevations at the borehole locations are referenced to Geodetic datum. The as-drilled borehole locations for the current investigation and supplemental boreholes from the two previous investigations, as well as the ground/ice surface elevations at the drilled locations and borehole depths (excluding ice/ water) are summarized below.

| Borehole | Location (m) | | Ground/ Ice Surface Elevation (m) | Borehole Depth (m) |
|---|---|---|---|---|
| | Northing | Easting | | |
| 4 | 5505329.6 | 331293.5 | 237.3 | 8.8 |
| 5 | 5505349.5 | 331229.3 | 237.3 | 10.1 |
| 102 | 5505345.2 | 331210.5 | 237.3 | 9.2 |
| 103 | 5505325.0 | 331307.7 | 237.3 | 10.2 |
| MA-1 | 5505342.1 | 331222.6 | 237.3 | 15.2 (below bridge deck)<br>11.5 (below existing ground surface) |
| MA-2 | 5505343.4 | 331249.6 | 231.2 | 9.9 |
| MA-3 | 5505330.5 | 331239.8 | 231.1 | 6.6 |
| MA-4 | 5505322.6 | 331268.1 | 231.3 | 7.0 |
| MA-5 | 5505337.4 | 331279.0 | 231.2 | 5.1 |
| MA-6 | 5505319.0 | 331291.6 | 235.9 | 9.6 |

# 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

## 4.1 Regional Geology

Published literature indicates that the site is located in the Quetico Subprovince of the Superior Province (Geology of Ontario; OGS Special Volume 4)[3]. The bedrock in a large area within and surrounding the Town of Hearst consists of muscovite-bearing granitic rocks (peraluminous), and may include biotite granite. Beyond the muscovite-bearing granitic boundary, bedrock consists of metasedimentary rocks.

Based on terrain mapping by the Ontario Geological Survey[4], the subsurface soils in the vicinity of the site consist of ground moraine deposits of clayey till.

## 4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the Record of Borehole and Drillhole sheets attached in Appendix A. The Record of Boreholes for the boreholes from PML and MTC are attached in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes at the bridge location is shown on Drawings 1 and 2.

In general, the subsoils at the structure site consist of embankment fill (sandy and/or clayey) and underlain by a very dense sand and silt till deposit with cobbles and boulders. Possible bedrock or a boulder was encountered at three boreholes within the river. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

### 4.2.1 Fill

Borehole MA-1 was advanced through the existing deck at the west end of the bridge at Elevation 237.3 m and encountered asphalt 115 mm thick over concrete 370 mm thick. From ground surface below the bridge deck on the front embankment slope at Elevation 233.6 m, Borehole MA-1 encountered a 1.3 m thick layer of silty clay fill material containing trace organics. Borehole 102 was advanced at the west approach within 20 m of the west end of the structure. The ground surface at that borehole location is Elevation 237.3 m and asphalt 100 mm thick was encountered over sand fill extending to 2.4 m below existing ground surface. Underlying the sand fill in Borehole 102, about 1.3 m of silt fill was encountered.

Borehole MA-6, advanced on the east embankment south side slope encountered a 0.5 m thick layer of sand and gravel fill from ground surface at Elevation 235.9 m overlying a 4.2 m thick layer of clayey silt to silty clay fill. Occasional pockets of organic material were noted within the cohesive fill in Borehole MA-6.

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[3] Geology of Ontario, 1991. Ontario Geological Survey, special Volume 4, Part 1. Eds P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott, Ministry of Northern Development and Mines, Ontario.

[4] Northern Ontario Engineering Geology Terrain Study, OGS Electronic Map, printed July 2011

Borehole 103 was advanced at the east approach within 20 m of the east end of the structure. The ground surface at the borehole location is Elevation 237.3 m and asphalt 100 mm thick was encountered over sand fill extending to 3.2 m below existing ground surface. Underlying the sand fill in Borehole 103, about 2.8 m of silty clay fill was encountered. Organic inclusions were reported within the silty clay fill in Borehole 103.

Borehole 4 was advanced through the east bridge deck and Borehole 5 was advanced through the west approach slab prior to the bridge rehabilitation. As the rehabilitation included the lowering of front embankment slope, the fill layer shown on the two borehole logs are not representative of the current subsurface information.

Two SPT 'N'-values measured within the sand fill in Boreholes 102 and 103 were 14 and 35 blows per 0.3 m of penetration indicating a compact to dense relative density and one 'N'-value measured in the silt fill in Borehole 102 was 8 blows per 0.3 m of penetration indicating a loose relative density. The clayey silt to silty clay fill encountered during the current and previous investigations was considered firm to hard based on 'N'-values measuring between 7 and 44 blows per 0.3 m of penetration.

Atterberg limits testing carried out on three samples of the clayey silt to silty clay fill material from the current investigation and one sample from the previous investigations indicate liquid limits ranging from about 28 to 50 percent and plastic limits ranging from about 16 to 23 percent, yielding plasticity indices from about 12 to 27 percent. The results of the Atterberg limits testing carried out on three samples from the current investigation (Boreholes MA-1 and MA-6) are shown on the plasticity chart on Figure C1 in Appendix C, and indicate that the deposit ranges from a clayey silt of low plasticity to a silty clay of intermediate plasticity.

A grain size distribution test was carried out on one sample of the clayey silt fill layer from the current investigation (Borehole MA-6) and the results are shown on Figure C2. From PML's investigation, two grain size distribution tests of the sand fill and one grain size distribution test of the silty clay fill were carried out and the results are shown on the respective borehole logs.

The natural water content measured on samples of the sand fill from the previous investigation are generally about 3 percent and on samples of the clayey silt to silty clay fill from the current and previous investigation ranged between 22 percent and 33 percent.

### 4.2.2 Organic Silt

Underlying the clayey silt to silty clay fill in Boreholes MA-6 and 103, a layer of organic silt was encountered at about Elevation 231.2 m and 231.3 m, respectively, and was interpolated to be about 0.8 m and 0.9 m thick, respectively.

Two SPT 'N'-values measured within the organic silt layer were 12 and 19 blows per 0.3 m of penetration indicating a compact relative density.

The natural water content measured on the two samples of the organic silt is 20 percent and 86 percent.

### 4.2.3 Sand and Silt (Till)

A deposit of grey sand and silt till was encountered below the fill material in Boreholes MA-1, MA-6, 4, 5, 102 and 103 and from the river bed below 0.5 m to 0.6 m of ice/water in Boreholes MA-2 to MA-5. Towards the west end of the site, the till was comprised of silt, sandy silt and silty sand (i.e. generally higher silt content) with gravel, cobbles and boulders. Towards the east end of the site, the till was comprised mainly of sand and silt or silty sand with gravel, cobbles and boulders (i.e. generally higher sand and gravel content). The top of this deposit was encountered between Elevation 233.6 m and 230.3 m. All boreholes were terminated within this deposit at practical refusal between Elevation 228.4 m and 222.1 m, except Boreholes MA-2 to MA-4 which may have extended into bedrock or terminated within a 1.5 m to 1.6 m boulder, as discussed in Section 4.2.4. The thickness of this deposit was between 2.6 m (Borehole 4) and 10.2 m (Borehole MA-1), although the deposit may not have been fully penetrated.

SPT 'N'-values measured within the till deposit ranged from 21 to greater than 100 blows per 0.3 m of penetration suggesting a compact to very dense relative density. Difficult casing advance was observed during drilling through most of the till deposit and in Boreholes MA-1 to MA-6, soil coring using an HQ or NQ core barrel was used to advance the borehole which is indicative of the presence of cobbles and boulders. A tri-cone bit was also used in Borehole MA-1 to advance the casing. Figure C3 presents photographs of the rock core samples retrieved during the current investigation. The gravel, cobbles and boulders generally consisted of metasediment; however, quartz and siltstone rock types were also recovered.

Grain size distribution tests were carried out on fifteen samples of the sand and silt till deposit from the current investigation and the results are shown on Figure C4. The samples were recovered from the standard 50 mm O.D. sampler and, as such, coarser grain sizes are not represented in these grading results. As noted above, the grain sizes range from silt to sand and gravel but, overall, the gradation envelope generally classifies this deposit as a sand and silt till. Furthermore, due to the nature of drilling using wash boring, finer particles may have been washed out leaving the sand/gravel sizes. The results of the grain size distribution tests from the previous investigations are shown on their respective borehole logs.

The natural water content measured on samples of the sand and silt till ranges from 2 to 19 percent.

### 4.2.4 Possible Bedrock or Boulder

Bedrock or a relatively large boulder (1.5 m to 1.6 m) was encountered and cored in Boreholes MA-2 to MA-4. Based on a review of the bedrock/boulder, the rock was described as fine grained, fresh, grey metasediment containing 1 mm to 300 mm thick white quartz veins as presented in the Record of Drillhole sheets in Appendix A and shown photographically on Figure C3. The Rock Quality Designation (RQD) measured on the core samples ranges from 97 percent to 100 percent indicating a rock mass of excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006). The Total Core Recovery (TCR) is 100 percent.

### 4.2.5 Groundwater Conditions

The water levels were noted during and after the drilling operations in the boreholes. In general, the soil samples taken in the boreholes were noted to be moist to wet. For the borehole drilled in the river, the samples were generally noted to be wet. In Boreholes MA-1 and MA-6, advanced on the west and east embankment slopes, the unstabilized water level after completion of drilling was 1.6 m (Elevation 232.0 m) and 7.1 m (Elevation 228.8 m) below the ground surface, respectively. The water level in the standpipe piezometers in Borehole MA-6 (with the screen installed within the sand and silt till deposit) on March 24, 2011, the day after installation, was measured at 5.1 m below existing ground surface corresponding to Elevation 230.8 m.

The ice/water level in the Mattawishkwia River was measured between Elevation 231.1 m and 231.3 m during the field investigation in March 2011. In November 2008, the water level of the river was measured by others at Elevation 231.5 m and the high water level (50 year) is reported to be Elevation 234.7 m.

Groundwater and river water levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

## 5.0 CLOSURE

The field personnel supervising the drilling program was Ms. Nicole Gould, EIT. This report was prepared by Mr. Ryan Hawkins and Mr. André Bom, P.Eng., and was reviewed by Ms. Sarah E. M. Coyne, P.Eng., an Associate with Golder. A quality control review of the report was provided by Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project.

# Report Signature Page

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# PART B

## FOUNDATION DESIGN REPORT

## HIGHWAY 11 MATTAWISHKWIA RIVER BRIDGE REPLACEMENT

## SITE 39W-33, HEARST, ONTARIO

## MINISTRY OF TRANSPORTATION, ONTARIO, GWP 164-98-00

# 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects for the proposed Highway 11 bridge structure over the Mattawishkwia River. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations at the site.

The interpretation and recommendations presented are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure and approach embankments at this site. As such, where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

## 6.1 General

The existing bridge carrying Highway 11 over Mattawishkwia River is a continuous, four-span reinforced concrete ‘T’ beam superstructure and asphalt wearing surface with an overall length of about 72 m and width of about 10 m. The structure, erected in 1938, has three piers supported by spread footings. Based on the 1982 rehabilitation drawing, the west and east pier footings are founded at Elevation 229.7 m and 228.8 m, respectively, corresponding to about 1 m and 2 m below the river level. The end spans are cantilevered such that there are no abutments. Rehabilitation of the bridge and approach embankments has been carried out on several occasions. Tilting of the east and west piers toward the river was first observed in 1956, as well as settlement and undermining of the approach slabs, reportedly likely as a result of pier footings that were improperly designed/constructed (i.e. too small/narrow). In 1967, the east pier was grouted, gabions were placed on the east front slope and the approach slabs were lengthened from 3 m to 6 m. The tilting movement continued in spite of the rehabilitation. The gabion baskets were noted to be “pushing” against the pier. In 1982, the gabion baskets were removed and a sheet-pile tie-back anchor wall was installed to lower the grade of the east and west front slopes. We understand that the sheet-pile wall was driven to Elevation 230.7 m and 229.6 m on the west and east sides of the bridge, respectively. A dead-man anchor system comprised of a concrete block “weight” located about 18 m behind the wall and “tie-rods” support the sheet pile wall laterally.

The proposed 73 m long, 14.5 m wide, three-span structure (30 m centre span and 21.5 m long end spans) will cross over the river at the same location as the existing bridge with a 1.4 m shift of the highway centreline to the north. The new structure will be 14.5 m wide and constructed in stages, such that the north side of the existing bridge can be removed following rerouting of both the westbound and eastbound traffic to a single lane on the south side of the existing bridge (i.e. eastbound lane). The wider structure and shifting of the highway centreline 1.4 m northerly will accommodate the routing of traffic to the new “half” structure. Once the new north “half” structure is completed, the south “half” of the existing bridge will be demolished allowing for the completion of the new structure.

The proposed grade at the west and east abutments will be Elevation 238.2 m and 237.8 m, corresponding to a grade raise of about 0.9 m and 0.6 m, respectively, and corresponding to embankments about 7 m high relative to the river bed. At the time of our field investigation (March 2011), the river was about 0.6 m deep (below ice surface) and the ice/water level was generally measured at Elevation 231.2 m. Reportedly, the water level surveyed by others in November 2008 was at Elevation 231.5 m and the 50-year high water level is at Elevation 234.7 m. The river is about 50 m wide at the structure location, depending on the water level.

The stratigraphic profile along the structure based on the results of several boreholes from the current and previous investigations is shown on Drawing 1. The stratigraphic profiles across the foundation areas are shown on Drawing 2. The subsoils generally consist of the existing pavement structure (asphalt over reinforced concrete at the bridge location and asphalt over sand fill beyond the bridge), embankment fill consisting of sand, silt and/or clayey silt to silty clay, underlain by dense to very dense sand and silt till. The till ranged in composition from being more silty towards the west end of the bridge and more sandy toward the east end of the bridge. The till was noted to contain sand and gravel layers as well as several occurrences of cobbles and boulders which were encountered during borehole advancement. Metasediment bedrock, or a large boulder having a size of between 1.5 m and 1.6 m or greater, was encountered at three of the boreholes located in the river (Boreholes MA-2 to MA-4).

Shallow foundations consisting of spread footings founded on the sand and silt till and deep foundations, comprised of steel H-piles or caissons founded within the sand and silt till, can be considered for supporting the proposed bridge abutments and piers. We understand from MMM that consideration is being given to an integral abutment structure requiring steel piles at the foundation elements for flexibility. Further, MTO discourages the use of mixed foundation types between the abutments and piers from a structural perspective. The advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives are summarized in Table 1, following the text of this report and discussion on the alternatives is presented below. Although steel H-piles and spread footings are ranked equivalent from a foundations perspective, we recommend the use of steel H-piles driven into the till or installed in 600 mm diameter pre-drilled holes for support of the bridge. For deep foundations, the shoring/dewatering requirements would be less costly than for shallow foundations and would also reduce the risk of negatively impacting the existing bridge during construction. However, caissons at the piers could eliminate the need for dewatering if extended to the bridge deck. Whichever foundation alternative is selected, the Contractor should be alerted to the very dense nature of and the presence of cobbles and boulders within the till material as it will impact shoring installation and foundation construction.

As the existing structure will be replaced in stages, consideration will need to be given to the existing sheet-pile tie-back anchor wall at each end of the bridge during excavation and construction in the vicinity of the new abutments. Temporary roadway protection will be required between the two travelled lanes to maintain the single lane of traffic during construction in the opposite lane. We understand from discussions with MMM that the Contractor will be alerted to the tie-back anchor system in the Contract Documents and it will be the Contractor’s responsibility to support the existing sheet-pile wall during the staged construction with excavations required in the area of the tie-back anchors. Foundation comments regarding the existing sheet-pile wall are provided in Section 6.9.4.

## 6.2 Steel H-Pile Foundations

The abutments and piers can be supported by steel H-piles driven into the sand and silt till to practical refusal. At the west abutment, Borehole MA-1 reached Elevation 222 m with difficulty advancing below Elevation 228 m. It is expected that the H-piles will advance below Elevation 228 m and that a pile length in excess of 5 m will be achieved. At the east abutment, Borehole MA-6 reached Elevation 226 m and it is considered that with hard driving the H-piles will reach this elevation and a pile length of 5 m will be achieved. However, depending on the location and depth of boulders within the till or the bedrock surface elevation at each pile location, some driven piles will likely not achieve a minimum pile length of 5 m. The piles will most likely hang up on or deflect off boulders at shallow depth (less than 5 m below bottom of foundation) and some additional piles would be required to accommodate the required structural loading. As an alternative to driving piles, in order to reduce the risk of piles hanging up and not reaching the required pile length, the piles could be installed in 600 mm diameter pre-drilled holes at each pile location. Specialized drilling equipment will likely be required for the pre-drilling due to the bouldery nature of the till.

For design, the estimated pile tip elevations are presented below. It should be noted that the pile lengths will vary if driven to refusal.

| Foundation Unit | Location within Foundation Unit | Borehole Number | Proposed Underside of Pile Cap[1] (m) | Pile Tip Elevation based on 5 m Pile Length (m) | Anticipated Pile Tip Elevation for Piles Driven to Practical Refusal[2] (m) |
|---|---|---|---|---|---|
| West Abutment | South | MA-1 | 233.2 | 228.2 | 227.0 |
| | North | 5 | | | 227.0 |
| West Pier | South | MA-3 | 229.5 | 224.5 | 226.5 |
| | North | MA-2 | | | 224.5 |
| East Pier | South | MA-4 | 229.5 | 224.5 | 227.0 |
| | North | MA-5 | | | 227.0 |
| East Abutment | South | MA-6 | 232.7 | 227.7 | 227.0 |
| | North | 4 | | | 227.0[3] |

Note: [1] Elevations shown on GA; at piers, founding elevation based on 1 m of soil cover below bottom of river, as further discussed in Section 6.2.3.
[2] Elevations based on results of the borehole drilling; varying driving depths will occur and will depend on driving equipment and presence of boulders at each pile location.
[3] Borehole 4 encountered refusal at Elevation 228.4 m; tip elevation assumed to be the same as Borehole MA-6.

At the abutments, pile driving without the need for pre-drilling may be feasible as the anticipated tip elevations for pile driving to practical refusal are below the minimum pile tip elevations for 5 m long piles. At the piers, the anticipated tip elevations for pile driving are higher than the elevation required to achieve a minimum 5 m long pile and, therefore, pre-drilling to Elevation 224. 5 m for installation of 5 m long piles will be required.

Dewatering will be required at the piers for construction of the concrete for pile cap construction in-the-dry, as discussed further in Section 6.9.3. The abutments will likely be located above the river/groundwater level, depending on the time of year in which construction takes place. Sub-excavation of the existing clayey fill below the west abutment and the organic silt below the east abutment will be required in advance of the abutment construction, as discussed in Section 6.7; the organic and clayey fill soils should be replaced with SP 110S13 Granular ‘B’ Type II.

If the piles are installed in pre-drilled holes, they will have to be fixed at the base in a sufficient depth of concrete (to be determined by the structural engineer) to achieve fixity of the lower section of the pile. As such, the pile tip elevations indicated above may have to be lowered or adjusted as required for structural considerations and depth of drilling indicated above may increase. Driving the piles to refusal after pre-drilling to avoid the use of concrete at the base of the pre-drilled holes may not be feasible as there may be very little penetration below the bottom of the pre-drilled holes resulting in minimal or no fixity for the bottom of the pile.

Pre-drilling will be required (at about Elevation 230 m) to install the 600 mm diameter corrugated steel pipes (CSPs) as part of the integral abutment design. The CSPs should be backfilled with loose, fine to medium sand. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is included in Appendix D.

At the abutments, specifically at the east abutment, the north half of the proposed structure is relatively close or within the footprint of the existing sheet pile wall. As further discussed in Section 6.9, the structural designer should check the location of new piles relative to the existing sheet pile wall and the Contractor should be alerted to the presence of the sheet pile wall.

### 6.2.1 Geotechnical Axial Resistance

For HP 310X110 piles driven to practical refusal in the till or installed within pre-drilled holes, the factored axial geotechnical resistance at ULS may be taken as 1,600 kN. The geotechnical resistance at SLS may be taken as 1,400 kN. For the abutment and pier piles, the pile driving note from the MTO Structural Manual (2008) is Note 1 which shall read:

- "Piles to be driven in accordance with standard SS 103-11 using an ultimate geotechnical resistance of 3,200 kN per pile."

Consideration could be given to the use of a heavier pile section to facilitate driving into the dense to very dense bouldery till. Factored geotechnical axial resistances at ULS of 1,800 kN and 2,200 kN should be used for HP 310X125 and HP 360X152 piles, respectively, corresponding to geotechnical resistances at SLS of 1,600 kN and 1,900 kN, respectively.

Piles driven into the till should be fitted with driving shoes and flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe) to minimize damage to the pile during driving and penetration through the cobbles and boulders overlying the bedrock.

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). Where piles are placed and not driven, pile driving notes/criteria are not required from the MTO Structural Manual (2008).

### 6.2.2 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The evaluation of the piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The lateral load response of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, $k_h$, (kPa/m) is determined in accordance with Section C6.8.7 in the Commentary to the CHBDC (2006) based on the equation for cohesionless soils given below (CFEM, 1992).

$$k_h = \frac{n_h z}{b}$$

where $n_h$ is the constant of horizontal subgrade reaction (kPa/m)
z is the depth below the underside of the pile cap (m)
b is the pile diameter or width (m)

It is understood that an integral abutment foundation design is being considered. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

For piles driven through the till to practical refusal (i.e. not pre-drilled holes), the lateral resistance of the piles will be developed primarily from the passive resistance of the soil over the portion of the piles below the CSP liners. The values of $n_h$ to be used to calculate the coefficient of horizontal subgrade reaction ($k_h$) to be utilized in the structural analysis for the piles where piles are installed within the CSP liners and driven through the till at this site are given below.

Loose Sand within CSP: $n_h$ = 2,200 kPa/m

Compact to very dense till (below the water level): $n_h$ = 11,000 kPa/m

For a single HP310X110 pile extending 2 m into the till below the 3 m CSP, the estimated factored lateral resistances at ULS is 70 kN and at SLS (for 10 mm of horizontal deflection at the pile cap) is 35 kN. These values are based on the commercially available program LPILE Plus (Version 5.0), produced by Ensoft Inc.

For piles installed in the CSP liners and in pre-drilled holes to the bottom of the pile, the lateral resistance of the piles will be developed primarily from the fixity (presumably in concrete) at the base of the pre-drilled holes. In this case, the structural resistance of the pile will govern the lateral resistance.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor, R, as follows:

| Pile Spacing in Direction of Loading d = Pile Diameter | Subgrade Reaction Reduction Factor |
|---|---|
| 8d | 1.00 |
| 6d | 0.70 |
| 4d | 0.40 |
| 3d | 0.25 |

The subgrade reaction reduction factor should be interpolated for pile spacings in between those listed above.

### 6.2.3 Frost Protection

The pile caps should be provided with a minimum of 2.6 m of conventional soil cover for frost protection, as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario). We understand that MMM is considering 1 m soil cover for the pier pile caps (below the riverbed), which will be appropriate provided the river does not freeze to the bottom at the pile locations as was the case in mid-March, 2011. The flowing water throughout the winter will prevent frost from penetrating to the subsoils below the pile cap. If it is possible that the river will freeze entirely during winter, we recommend that the pile caps be provided with 2.6 m of soil cover.

## 6.3 Shallow Foundations

Consideration could be given to supporting the abutments and piers on spread footings placed directly on the native compact to very dense till material. The recommended minimum founding elevation at each foundation element for the footing is summarised below.

| Foundation Element | Location within Foundation Unit | Borehole Number | Recommended Founding Elevation on Till (m) |
|---|---|---|---|
| West Abutment | South | MA-1 | 231.5 |
| | North | 5 | |
| West Pier | South | MA-3 | 229.5 |
| | North | MA-2 | |
| East Pier | South | MA-4 | 229.5 |
| | North | MA-5 | |
| East Abutment | South | MA-6 | 230.0 |
| | North | 4 | |

For a higher footing elevation at the abutments, a granular pad would be required extending from the proposed footing elevation to the elevations given in the table above. For footing elevations similar to the underside of pile caps shown in the GA (Elevation 233.2 m and 232.7 m at the west and east abutments, respectively), the granular pad would be between about 2 m and 3 m thick. The granular pad should be designed and constructed as described in Section 6.9.6.

Dewatering will be required at the abutments and piers to allow for placement of the concrete for footing construction in-the-dry or for placing and compacting the granular pad in-the-dry, as discussed further in Section 6.9.3.

### 6.3.1 Geotechnical Resistance

For spread footings placed directly on or below the surface of the properly prepared compact to very dense sand and silt till at or below the elevations specified above, or on a minimum 2 m thick compacted granular pad, a factored geotechnical axial resistance at ULS of 600 kPa may be used for design. A corresponding SLS value of 350 kPa may be used assuming 2 m to 3 m wide footings.

All loose, softened or disturbed till material at the subgrade elevation should be removed and replaced with mass concrete. Construction and inspection of footings should be carried out in accordance with Ontario Provincial Standard Specification (OPSS) 902 (Excavating and Backfilling – Structures).

The geotechnical resistances provided above are given for the condition that the loads are applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the CHBDC and its Commentary.

### 6.3.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the concrete footings and the till or the compacted granular pad should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan $\delta$, may be taken as 0.55 between the base of the concrete footings and the till and as 0.60 between the concrete footings and the granular pad (NAVFAC, 1982) for construction in-the-dry.

### 6.3.3 Frost Protection

Footings constructed on the till or over a granular pad should be provided with a minimum of 2.6 m of soil cover or equivalent thickness of insulation for frost protection (OPSD 3090.100 Foundation Frost Penetration Depths for Northern Ontario). At the piers, as discussed in Section 6.2.3, we understand MMM is considering a founding level at 1 m below ground surface (river bed). Spread footings founded at 1 m below the river bed should be suitable provided the river does not freeze thereby preventing frost penetration to the footing subgrade elevation. If it is possible for the river to freeze completely, then the full soil cover should be provided.

## 6.4 Caissons

Consideration could be given to the use of caissons for support of the piers. Caissons are not typically as economically feasible as piles and the larger diameter holes for caissons (1.0 m, 1.5 m or greater) relative to the smaller pre-drilled holes for steel H-piles may be more difficult to install due to cobbles and boulders present within the till. Caissons may be more advantageous than steel H-piles as the piers may extend to the underside of the bridge deck without the need for a pile cap which would require a cofferdam and dewatering. Further, the high axial capacity of the caissons would result in fewer units being required to support the abutments than that required for the H-pile design.

Temporary or permanent steel liners and tremie concrete will be required to install caissons at this site. Further, full balanced head conditions should be maintained at all times during construction to reduce the potential for base heave in the bottom of the caissons and ground loss, which could potentially cause settlement of the shallow foundation of the adjacent bridge structure.

#### 6.4.1.1 Geotechnical Axial Resistance

If caissons are considered as a founding alternative at the pier, the caissons at this site will derive their axial resistance mainly from the shaft resistance within the bedrock below Elevation 223.4 m at the west pier (Borehole MA-2) and below Elevation 225.4 m at the east pier (Borehole MA-4). The contribution from end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of the sockets. The factored geotechnical axial resistance at ULS for two different caisson sizes, both socketted a minimum of 2 m into the bedrock are given below:

| Caisson Diameter (m) | Minimum 2 m Socket at West and East piers below Elevation 223.4 m and 225.4 m, respectively | |
|---|---|---|
| | ULS (kN) | SLS for 25 mm |
| 1.0 | 5,000 | n/a |
| 1.5 | 7,000 | n/a |

The resistance required to achieve 25 mm of settlement is greater than that given for ULS for caissons socketted into the bedrock and, therefore, SLS conditions do not apply.

#### 6.4.1.2 Resistance to Lateral Loads

The lateral load response of a single caisson is dependent on the strength of the concrete and the steel reinforcement details (i.e. location, quantity, diameter, etc.). As the caisson details are unknown at this time, estimated maximum lateral resistances for caissons could be provided upon request under separate cover.

#### 6.4.1.3 Frost Protection

If required, the pile caps for the caissons in the river should be provided with a minimum of 1 m of conventional soil cover for frost protection as described in Section 6.2.3.

## 6.5 Seismic Considerations

### 6.5.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site, based on experience and considering the guidelines in Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

### 6.5.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the CHDBC. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Performance Zone 0. The site-specific zonal acceleration ratio for the Hearst area is 0.00. Based on experience, for the subsurface conditions at this site, no amplification of the ground motion is recommended for design (i.e. Site Coefficient, S = 1.0).

We understand, based on Section 4.4.4 of the CHBDC, that this bridge structure is assigned Seismic Performance Zone 0. Given this, and in accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Performance Zone 0.

## 6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.5.2, seismic (earthquake) loadings need not be analyzed for this structure.

The following recommendations are made concerning the design of the abutment walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of SP 110S13 (Aggregates) Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with SP 105S10 (Compaction). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill) and 3121.150 (Walls, Retaining, Backfill).

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for "Backfill to Structures Adjacent to Rock Embankment Approaches, dated November 2002 (and most recent SP and OPSD as referenced herein). Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill Rock).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.

- For restrained structures, the granular fill should be placed in a zone with width equal to at least 2.6 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC).

- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

| | Earth Fill | Rock Fill |
|---|---|---|
| Soil unit weight: | 21 kN/m$^3$ | 19 kN/m$^3$ |
| Coefficients of static lateral earth pressure: | | |
| Active, $K_a$ | 0.31 | 0.22 |
| At rest, $K_o$ | 0.47 | 0.35 |

- For unrestrained structures, the pressures are based on the rock fill as above or on the granular fill as placed and the following parameters (unfactored) may be assumed:

| | Granular 'A' | Granular 'B' Type II |
|---|---|---|
| Soil unit weight: | 22 kN/m$^3$ | 21 kN/m$^3$ |
| Coefficients of static lateral earth pressure: | | |
| Active, $K_a$ | 0.27 | 0.27 |
| At rest, $K_o$ | 0.43 | 0.43 |

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHDBC).

A restrained structure is typically a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

## 6.7 Approach Embankment Design and Construction

The new bridge will be replaced along the same alignment as the existing bridge with a final grade of Elevation 238.2 m and 237.8 m at the west and east abutments, respectively, resulting in grade raises of 0.9 m and 0.6 m at these locations, respectively. Further, the new bridge will be wider by about 4.5 m and the new centreline shifted to the north of the existing centreline by 1.4 m, resulting in embankment widening of less than 4 m to the north and less than 1 m to the south. The new embankments will be approximately 7 m above the river bed.

The soils encountered within the footprint of the proposed approach embankments consist of clayey fill, organic silt (east approach) overlying compact to very dense sand and silt till with cobbles and boulders.

For the purpose of analysis, granular fill has been considered for the construction of the approach embankments using side slopes at 2H:1V. Should an economical source of rock fill become available for this site, side slopes should be formed at 1.25H:1V although we understand from MMM that there is no such source and, therefore, rock fill has not been considered in the analysis.

The stability and settlement analyses of the new approach embankments are focused on the critical sections, which consist of the front and north (i.e. widened) side slopes. As there is minor widening at the south side of the bridge (i.e. less than 1 m), stability and settlement of the south embankment side slopes for the approach embankment are less of a concern. During the analysis, we have assumed that the existing fill and organic silt (east approach) within the existing embankment remains in place, but that the organic silt is removed beneath the footprint of the new widened embankment.

Sections 6.7.1 and 6.7.2 of this report summarize the methods used to analyze the stability and settlement of the approach embankments. For design purposes, the groundwater level is assumed to be equivalent to the river water level measured in March 2011 at Elevation 231.2 m. The analyses have also been checked against the reported high water level of Elevation 234.7 m.

During staged construction of the new structure, consideration should be given to the stability of the existing sheet-pile wall and dead-man anchor systems which retain the existing embankment front slopes. Temporary roadway protection is required behind the abutments along the existing embankment to construct the new embankments and foundations using staged construction methods (see Section 6.9.3).

### 6.7.1 Stability

The following section outlines the methodology and presents the parameters used to evaluate embankment stability at the west and east approach embankments. The geometry of the proposed west and east approach embankments used in the analysis is based on the information from the GA and the analyzed geometry for the front slopes and north side slopes is shown on Figures 1 to 4.

#### *6.7.1.1 Methodology*

The limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.17), produced by Geo-Slope International Ltd., employing the Morgenstern Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions for MTO sites. This FoS is considered adequate for the embankments at these sites considering the design requirements and the field data available and is based on deep seated, global failure surfaces that would affect the operation of the roadway. The stability analyses were performed to check that the target minimum FoS was achieved for the proposed embankment height and geometry.

#### *6.7.1.2 Parameter Selection*

For the native sand and silt till, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle and effective cohesion) for the soil were estimated from empirical correlations using the results of in situ SPT, in conjunction with engineering judgement based on experience in similar soil conditions.

The simplified stratigraphy together with the associated strength and unit weight employed for the different soil types are summarized below.

| Soil Type | Unit Weight ($kN/m^3$) | Angle of Internal Friction ($^o$) |
|---|---|---|
| Existing Fill | 20 | 30 |
| Organic Silt (East Approach) | 19 | 28 |
| New Granular Fill | 21 | 35 |
| Sand and Silt (Till) | 21 | 35 |

### *6.7.1.3 Results of Stability Analysis*

The stability analysis performed on the front slopes and north side slopes for the west and east approach embankments indicates that after the completion of embankment construction, the FoS would be greater than 1.3 and that stability mitigation is not required. The results for the west and east front and north side slopes are shown in Figures 1 to 4.

## 6.7.2 Settlement

Settlement of the new approach embankments can be expected as a result of the loading from the new fills on the foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the embankment fill itself; however, typically for well compacted granular fill (i.e. sand and gravel), settlement of the new granular embankment fill is minimal. To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using hand and spreadsheet calculations.

### *6.7.2.1 Settlement Criteria*

Based on MTO's "Embankment Settlement Criteria for Design" Final Draft dated March 2, 2010, the following post-construction settlement and differential settlement criteria are considered acceptable to occur within 20 years post-paving for the bridge approach embankments and the new embankments at this site.

| Location | Distance from Transition Point (i.e. Abutment) | Total Post-Construction Settlement (mm) |
|---|---|---|
| Transition/Taper to Bridge Abutments | 0 m to 20 m | 25 |
| | 20 m to 50 m | 50 |
| | 50 m to 75 m | 75 |

In addition to the above, differential settlement across the embankment should not exceed 100:1 for non-freeways.

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

#### 6.7.2.2 Parameter Selection

The immediate compression of the existing fill and native cohesionless subsoils were assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Kulhawy and Mayne (1990). The simplified stratigraphy, unit weights and deformation parameters employed for the different soil types in the approach areas are summarized below.

| Material | Location | Approximate Thickness at Centreline of Embankment | Unit Weight (kN/m$^3$) | Estimated Deformation Properties |
|---|---|---|---|---|
| New Granular Fill | West | 0.9 m | 21 | - |
| | East | 0.6 m | | |
| Existing Fill and Organic Silt (East Approach) | West | 3.7 m (Borehole 102) | 20 | E' = 10 MPa |
| | East | 6.8 m | | |
| Sand and Silt Till | West | 5 m (to top of boulder layer) | 21 | E' = 30 MPa |
| | East | 3 m (to top of boulder layer) | | |

The maximum estimated settlement of the foundation soils in these areas (due to the loading imposed by the new approach embankment fill) is presented below and a discussion on the rate of settlement is included.

#### 6.7.2.3 Results of Settlement Analysis

Settlement of the new granular fill will be less than 25 mm if placed and compacted properly (see Section 6.8). The estimated magnitude of settlement of the existing fill, preloaded organic silt (east approach) and native foundation soils below the new embankment fill is also expected to be less than 25 mm. The settlement of the new and existing fill and native soils will occur rapidly (during construction).

Since the post-construction settlement of the new embankment within 20 m of the abutments is generally less than the settlement criteria referenced in Section 6.7.1.1 (i.e. 25 mm within 20 m of the bridge abutment), settlement mitigation is not required.

Depending on the construction schedule, minor differential settlement (i.e. 25 mm or less) across the embankment perpendicular to the highway alignment may occur between the existing and widened section of the embankment, which may fall directly within the new westbound lane of the highway.

## 6.8 Subgrade Preparation and Embankment Construction

For the bridge approach embankments within 20 m of the abutments, removal of the existing organic and clayey fill is required below the widened embankment footprint prior to placement of new fill and construction of the new abutments. All softened/loosened soils should also be stripped from below the approach embankments, prior to placement of new fill. The backfill in the frost taper zone should be constructed in accordance with OPSD 3101.150 (Walls, Abutment, Backfill Minimum Granular Requirement).

Granular fill materials and placement should be in accordance with the requirements as outlined in SP 206S03 (Earth Excavation, Grading). Granular fill placed below the water level should consist of SP 110S13 Granular 'B' Type II. All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density. Prior to placement of the pavement granular subbase and base courses, the final lift of embankment fill should be compacted to not less than 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The new fill should be keyed into the existing embankment side slopes per the requirements of OPSD 208.010 (Benching of Earth Slopes).

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 (Rip Rap, Granular Sheeting) and SP 511S01 (Rip Rap Gravel Sheeting). Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (300 mm diameter as per OPSS 1004), rock protection or concrete slope paving. The designer should address the potential for scour below the pile caps in the design of the bridge foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS 804 (Seed and Cover) should be carried out as soon as possible after construction where earth fill is used. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with an erosion control blanket, straw, or gravel sheeting as per OPSS 511 (Granular Sheeting) to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

## 6.9 Design and Construction Considerations

### 6.9.1 Excavations

Prior to abutment construction, sub-excavation will be required to remove organic material from the footprint of the new widened embankment and sub-excavation of the existing fill will be required for abutment construction. The underside of the west and east abutments are at Elevation 233.2 m and 232.7 m, respectively, resulting in excavations up to 4 m and 5 m below the existing ground surface, respectively. The excavations will likely be above the water level at the abutments depending on the river level at the time of construction.

As the structure will be replaced in stages with traffic reduced to one lane in the vicinity of the bridge, the excavations at the abutments and approach embankments will be supported by temporary shoring system to maintain the stability of the existing roadway embankment. This will likely be accomplished using a sheet-pile cut-off wall or a soldier pile and lagging wall. Further discussion and recommendations regarding temporary shoring is provided in Section 6.9.3.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects and good construction practice. The existing fill materials should be classified as Type 3 soil and the native sand and silt till should be considered a Type 2 according to the OHSA.

### 6.9.2 Subgrade Protection

If spread footings are considered for this site, the very dense sand and silt till subgrade is susceptible to disturbance from construction traffic (machine and foot), ponded water, etc., once exposed. Following excavation to the subgrade level, the exposed subgrade (i.e. till) should be protected from machine and foot traffic and weather by the use of a 5 MPa lean concrete "working slab" placed within 4 hours of first exposure and after review by the Contractor's Quality Verification Engineer (QVE). The working slab should be a minimum of 100 mm thick to limit the disturbance and to provide a platform for construction of the spread footing. The Contractor should be aware that trafficking over the exposed silty material may not be possible and an NSSP for placement of the working slab and protection of the subgrade should be contract specifications; an example is included in Appendix D.

### 6.9.3 Groundwater and Surface Water Control

Although perched water was not encountered within the fill during the investigation, it is possible that water is perched within the fill materials, and the Contractor should anticipate perched water within the fill sub-excavation for the bridge approach embankments. Surface water should be directed away from the excavation at all times.

As the underside of the west and east abutments will likely be located above the river water level, depending on the season and the amount of precipitation during construction, the pile caps will likely be constructed in-the-dry with dewatering by pumping from sumps, as required.

Where organics are encountered below the water level beneath the new widened embankment footprint, the organics below water level could be sub-excavated in thin strips and replaced with Granular 'B' Type II while the excavation advances, in accordance with OPSS 209.

At the piers, excavations for pile cap construction will be advanced below the river bed into the water-bearing sand and silt till and appropriate dewatering of these deposits (i.e. within a temporary cofferdam) will be required to maintain the water level below the founding level during excavation and construction. It is recommended that an NSSP be included in the Contract to address dewatering for the pier pile cap construction; a example NSSP is included in Appendix D. The Contractor should also be alerted that excavations for the pier foundations will be advanced through cohesionless soils, which may be unstable below the water level at this site.

As driving steel sheeting through the river bed will be impractical due to the presence of cobbles and boulders, the following excavation and temporary shoring procedures may be considered for pier foundation construction some 1 m to 2 m below river level:

- Excavation into the river bed within a rubber dam or within a temporary box, lowering the box as the excavation advances;

- Granular ‘B’ Type II will be placed at the bottom of the excavation within the box followed by pre-drilling the 600 mm diameter holes and pile installation/driving;
- Tremie concrete will be placed over the Granular ‘B’ Type II to the founding elevation; and
- Dewatering within the temporary box for foundation construction in-the-dry.

From a foundations perspective, the above construction methods will be feasible for constructing the piers in-the-dry. Regarding excavations within the river bed, the Contractor should be alerted to the presence of cobbles and boulders as discussed in Section 6.9.5.

If caissons are used to support the bridge at the piers and are extended to the bridge deck, then the need for dewatering at these locations may be reduced/eliminated.

### 6.9.4 Temporary Shoring and Existing Sheet-Pile Tie-Back Anchor System

As part of the design and construction of the new abutment foundations, careful consideration should be given to the location of the new piles relative to the existing sheet-pile wall (tied back with deadman anchors) and proposed temporary shoring required between the eastbound and westbound lanes for staged construction. Specifically, the designer should check that the new piles (batter and orientation) and temporary shoring do not interfere with the existing structure and tie-back anchor system. This should be checked for the full extent of the pile/shoring length. MMM has indicated that it will be the Contractor’s responsibility to support the existing sheet-pile wall during the staged construction with excavations required in the area of the tie-back anchors.

Temporary excavation support systems at the site should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). As the proposed temporary shoring will likely be used to support the existing sheet-wall laterally away from the shoreline, the lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539 (Temporary Protection Systems).

### 6.9.5 Obstructions

The Contractor should be alerted to the tie-back anchor system and the sheet pile wall in the Contract documents. The Contractor should also be alerted to the presence of cobbles and boulders. An example Operational Constraint alerting the Contractor to these obstructions is included in Appendix D.

### 6.9.6 Granular Pad for Spread Footings

Should spread footings be considered for supporting the abutments, a compacted granular pad should be constructed such that it extends 1 m beyond the edge of the abutment and sloped no steeper than 1H:1V and slopes. We recommend the pad be constructed using SP 110S13 Granular ‘B’ Type II and in accordance with SP 206S03 (Earth Excavation, Grading).

### 6.9.7 Monitoring of Existing Structure

We recommend that the existing structure as well as the steel sheet-pile wall at the front slopes be monitored for settlement and lateral movement while excavation, pile driving or other construction activities are carried out at the site given the following:

- The age and rehabilitation history of the existing structure;
- The requirement for operation of the south half of the existing structure during construction of the new north half of the proposed structure; and
- The proximity of the existing and new foundation elements, specifically the abutments.

The type, location and number of settlement and lateral movement points and frequency of readings should be developed by the bridge designer.

## 7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., an Associate with Golder. A quality control review of the report was provided by Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project.

# Report Signature Page

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Fintan J. Heffernan, P.Eng.
Designated MTO Contact

AB/SEMC/FJH/lb

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N:\Active\2010\1190 Sudbury\1191\10-1191-0038 MMM Hwy 11 Mattawishkwia River\Reporting\Final Report\Mattawishkwia\10-1191-0038-R01 Final RPT 11Nov14 Mattawishkwia.Docx

## REFERENCES

Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, Fourth Edition.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6-06, 2006. CSA Special Publication, S6.1-06. Canadian Standard Association.

Ministry of Transportation, Ontario, 2008. Structural Manual. Provincial Highways Management Division, Highway Standards Branch, Bridge Office.

Ministry of Transportation, Ontario, 2001. Ontario Traffic Manual, Book 7. Temporary Conditions, Field Edition.

Ministry of Transportation Ontario Embankment Settlement Criteria for Design, Final Draft, March 2, 2010

Ministry of Transportation, Ontario. Backfill of Structures Adjacent to Rock Embankment Approaches, Northern Region Directive, November 2002.

Ministry of Transportation, 1998, Northern Region Embankment Design Guidelines, Northern Region Engineering Directive, NRE, pp 98-200, issued by Geotechnical Section, October 28, 1998.

Occupational Health and Safety Act and Regulation for Construction Projects, January 2006.

ASTM International

| | |
|---|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
| ASTM D1587 | Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes |
| ASTM D2573 | Standard Test Method for Field Vane Shear Test in Cohesive Soil |

Commercial Software

LPILE PLUS (Version 5.0) by Ensoft Inc.

GeoStudio (Version 7.17) by Geo-Slope International Ltd.

Ministry of Transportation Ontario Special Provisions

| | |
|---|---|
| SP 110S13 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |
| SP 206S03 | Earth Excavation, Grading |
| SP 511S01 | Rip Rap; Rock Protection, Gravel Sheeting |

Ontario Provincial Standard Drawings

| | |
|---|---|
| OPSD 208.010 | Benching of Earth Slopes |
| OPSD 3000.150 | Foundation Piles, Steel H-Pile Splice |
| OPSD 3090.100 | Foundation, Frost Penetration Depths for Northern Ontario |
| OPSD 3101.150 | Walls Abutment, Backfill Minimum Granular Requirement |
| OPSD 3101.200 | Walls Abutment, Backfill Rock |
| OPSD 3121.150 | Walls Retaining, Backfill Minimum Granular Requirement |

Ontario Provincial Standard Specifications

| | |
|---|---|
| OPSS 501 | Construction Specification for Compacting |
| OPSS 511 | Construction Specification for Rip Rap, Rock Protection, and Granular Sheeting |
| OPSS 539 | Construction Specification for Temporary Protection Systems |
| OPSS 802 | Construction Specification for Topsoil |
| OPSS 804 | Construction Specification for Seed and Cover |
| OPSS 903 | Construction Specifications for Deep Foundations |
| OPSS 1002 | Material Specification for Aggregates – Concrete |
| OPSS 1004 | Material Specification for Aggregate - Miscellaneous |

Ontario Provincial Standard Structural Drawings

| | |
|---|---|
| SS 103-11 | Pile Driving Control, 2002 |

**Table 1: Evaluation of Bridge Foundation Alternatives**

| Options | Ranking | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| Steel H-Piles driven into very dense till or installed within 600 mm diameter pre-drilled holes | 1<br>(abutments and piers) | ■ Reduced size of excavations for pile caps compared with spread footings. This will be advantageous when considering shoring systems (i.e. cofferdams, etc.) and protection of deadman anchors.<br>■ Allows for integral abutment construction. | ■ Pre-augered holes through bouldery till may be required to extend piles to sufficient minimum length at the piers.<br>■ High possibility of piles “hanging” up on a cobble/boulder within the till deposit (if not installed in pre-drilled holes) including alignment and/or location concerns.<br>■ Dewatering required for pile cap construction within shored excavation; bouldery till will cause difficulties advancing/installing shoring. | ■ Installing 600 mm diameter CSPs into very dense till is expensive.<br>■ Lower relative costs compared with caisson option.<br>■ Costs for shoring and dewatering potentially less than for shallow foundation option but more than for caisson option if used at piers and extended to bridge deck (i.e. no pile cap). | ■ Risk of difficulties installing shoring and dewatering system.<br>■ Risk of not achieving minimum required pile length without pre-drilling.<br>■ Drilling/piling vibrations may negatively impact existing structure.<br>■ Monitor existing structure during shoring and pile installation. |
| Spread Footing founded on till deposit | 2<br>(abutments)<br>3<br>(piers) | ■ Relatively straightforward construction.<br>■ Reduces potential negative impact on existing bridge due to vibrations from piling/drilling. | ■ Larger excavations for spread footings compared with pile cap excavations, which will increase shoring/dewatering requirements.<br>■ Allows only for semi-integral abutment design.<br>■ Dewatering required for spread footing construction within shored excavation; bouldery till will cause difficulties advancing/installing shoring. | ■ Lower relative costs compared with piled foundations.<br>■ Costs for shoring and dewatering potentially higher than deep foundation options. | ■ Risk of difficulties installing shoring and dewatering system.<br>■ Monitor existing structure during excavations and shoring installation. |

| Options | Ranking | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| Caissons socketted into very dense till | 3 (abutments)<br>1 (piers) | ■ Reduced number of deep elements compared to steel H-piles (higher axial resistance per unit).<br>■ Possible elimination of pile cap (at piers) which would reduce the need for shoring/dewatering in the river. | ■ Difficulty advancing larger diameter caissons (compared to 0.6 m diameter pre-drilled holes for piles) through bouldery till deposit.<br>■ Temporary liners may be required for ground support during caisson advance.<br>■ Concrete for caissons would have to be placed by tremie methods below the water level. | ■ Relative costs higher than piles or spread footings.<br>■ Cost of shoring and dewatering may be reduced/eliminated at piers if caissons extend to bridge deck. | ■ Risk of difficulties penetrating the bouldery till.<br>■ Drilling vibrations may negatively impact existing structure.<br>■ Monitor existing structure during caisson construction.<br>■ Risk of difficulties installing shoring and dewatering system. |

MINISTRY OF TRANSPORTATION, ONTARIO

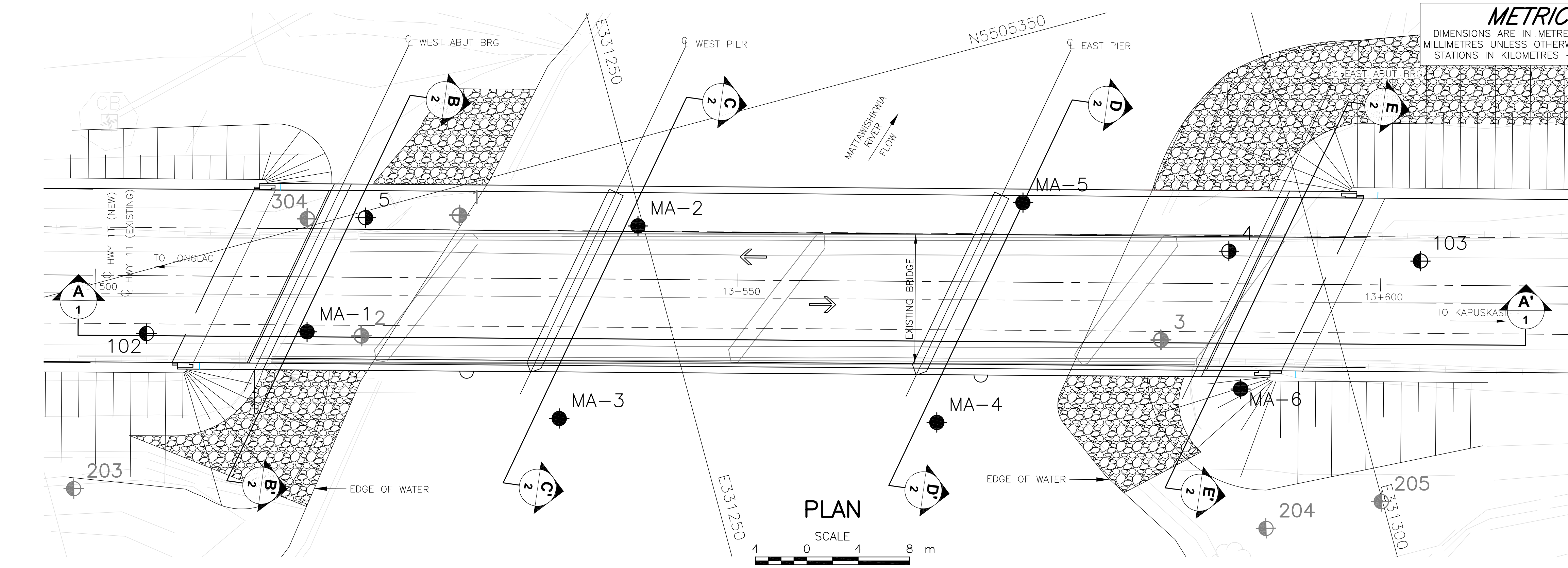
**METRIC**
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 164–98–00

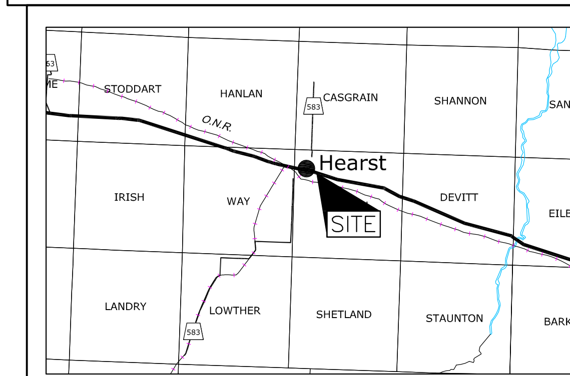
SHEET

# HIGHWAY 11
MATTAWISHKWIA RIVER BRIDGE
## BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA




PLAN
SCALE 4 0 4 8 m




KEY PLAN
SCALE 10 0 10 km

## LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (PETO, 2008)
- Borehole – Previous Investigation (MTC, 1978)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- R Refusal
- WL in piezometer, measured on MAR 24, 2011
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

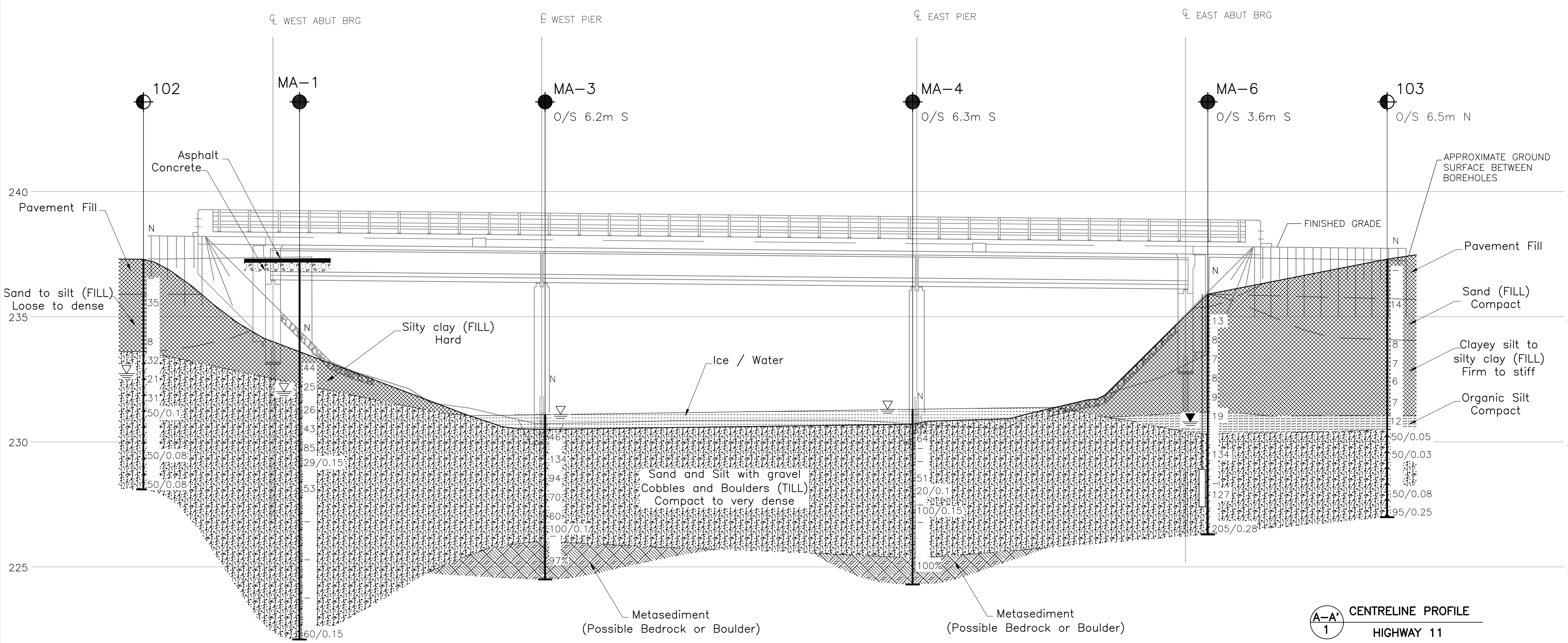
| No. | ELEVATION | NORTHING | EASTING |
|---|---|---|---|
| 4 | 237.2 | 5505329.6 | 331293.5 |
| 5 | 237.2 | 5505349.5 | 331229.3 |
| 102 | 237.3 | 5505345.2 | 331210.5 |
| 103 | 237.3 | 5505325.0 | 331307.7 |
| MA–1 | 237.3 | 5505342.1 | 331222.6 |
| MA–2 | 231.2 | 5505343.4 | 331249.6 |
| MA–3 | 231.1 | 5505330.5 | 331239.8 |
| MA–4 | 231.3 | 5505322.6 | 331268.1 |
| MA–5 | 231.2 | 5505337.4 | 331279.0 |
| MA–6 | 235.9 | 5505319.0 | 331291.6 |

## NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.




CENTRELINE PROFILE
HIGHWAY 11

HORIZONTAL SCALE 4 0 4 8 m
VERTICAL SCALE 2 0 2 4 m

A.J.K. BOM 100075715 NOV 14, 2011 PROVINCE OF ONTARIO
F. J. HEFFERNAN NOV 14, 2011 PROVINCE OF ONTARIO

## REFERENCE

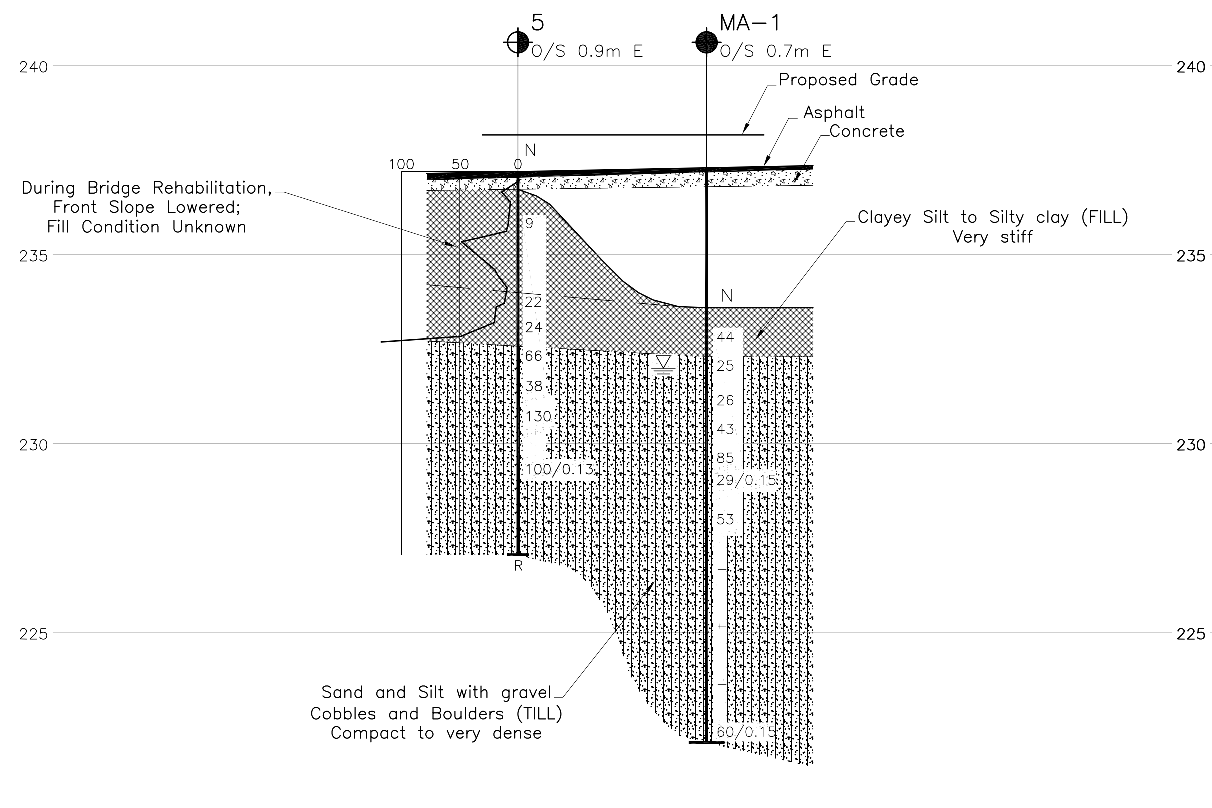
Base plan provided in digital format by MMM, drawing file no. P1–GENERAL ARRANGEMENT.dwg, dated AUG 2011, received SEPT 23, 2011. Key plan provided in digital format by MMM, drawing file no. Key Map 164–98–00.dwg, recieved JULY 15, 2011.

| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |

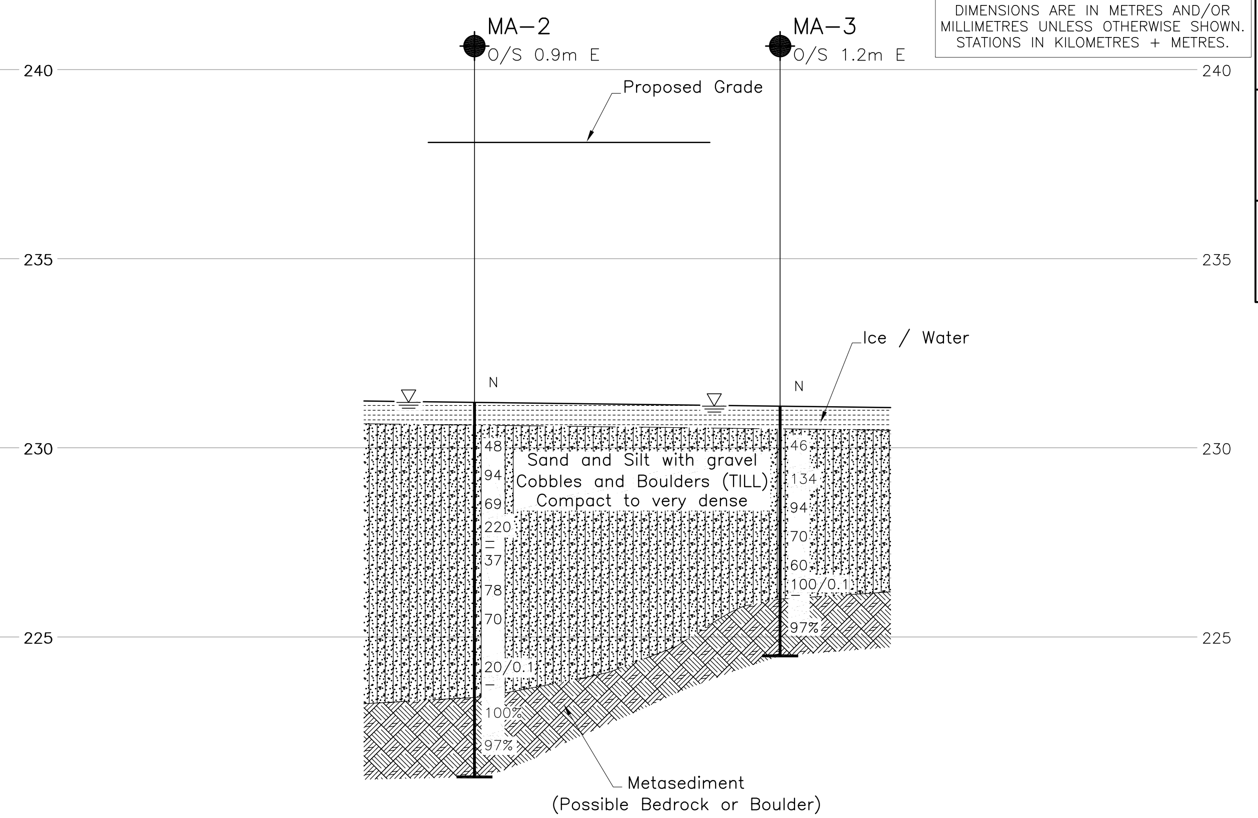
| | | | |
|---|---|---|---|
| Geocres No. 42G–34 | | | |
| HWY. 11 | PROJECT NO. 10–1191–0038 | | DIST. |
| SUBM'D. | CHKD. AB | DATE: NOV 2011 | SITE: 39W–33 |
| DRAWN: JJL | CHKD. SEMC | APPD. FJH | DWG. 1 |

PLOT DATE: November 14, 2011
FILENAME: \\sud1-s-cads\Golder\CAD\Projects\2010\10-1191-0038 MMM HWY 11 Mattawishkwia\10-1191-0038A001.dwg

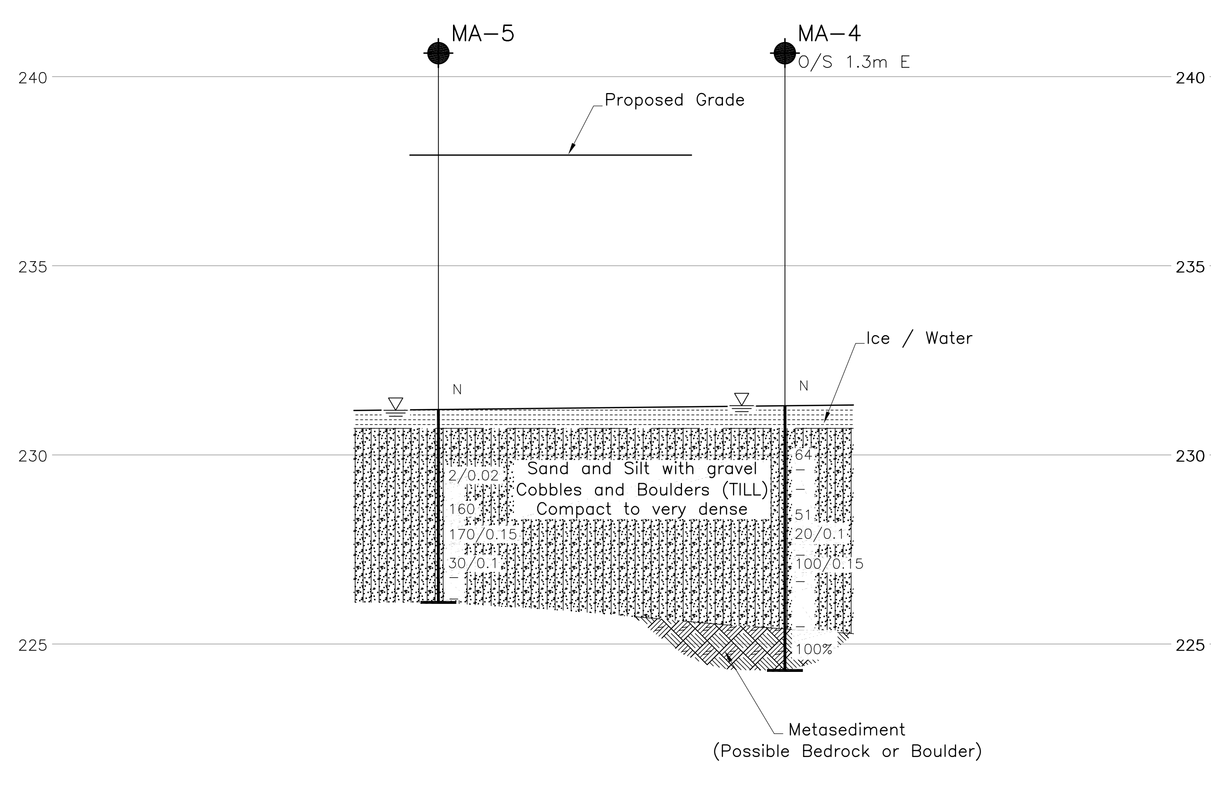
MINISTRY OF TRANSPORTATION, ONTARIO



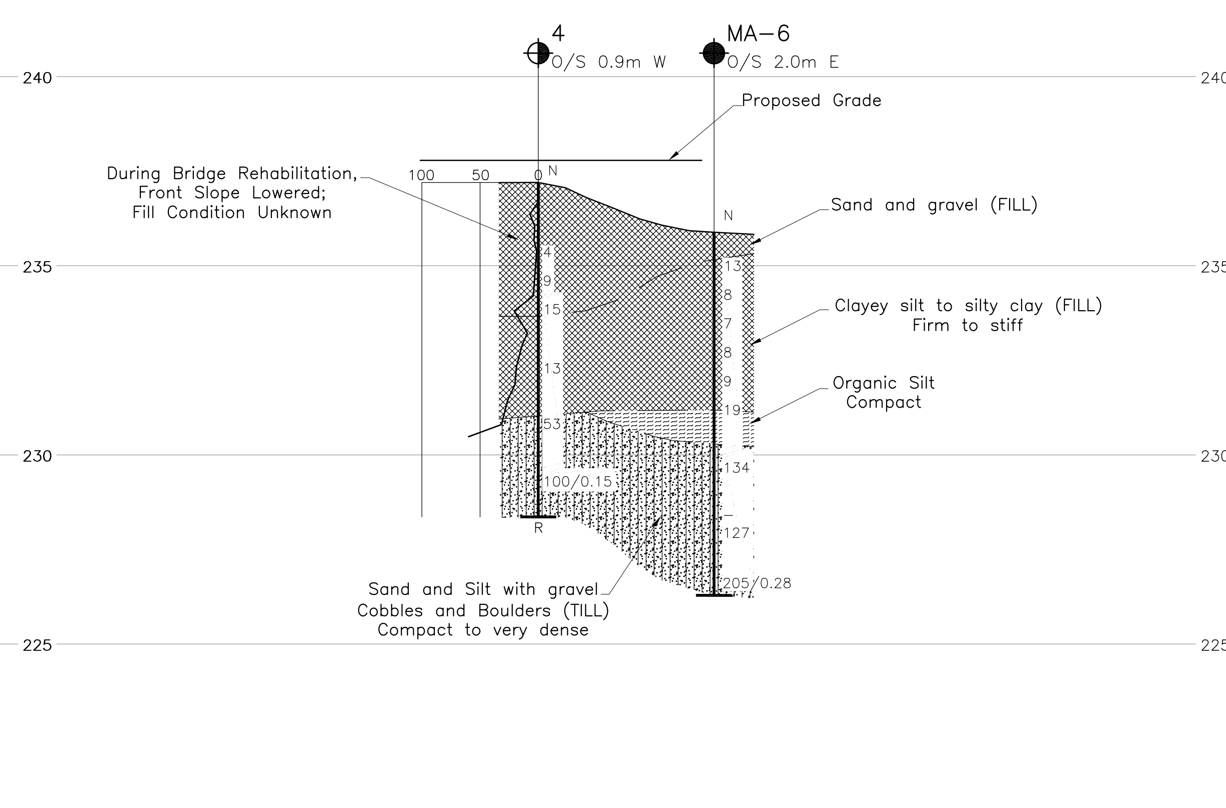

B-B' 2 WEST ABUT. SECTION
HIGHWAY 11
HORIZONTAL SCALE 4 0 4 8 m
2 0 2 4 m VERTICAL SCALE




C-C' 2 WEST PIER SECTION
HIGHWAY 11
HORIZONTAL SCALE 4 0 4 8 m
2 0 2 4 m VERTICAL SCALE




D-D' 2 EAST PIER SECTION
HIGHWAY 11
HORIZONTAL SCALE 4 0 4 8 m
2 0 2 4 m VERTICAL SCALE




E-E' 2 EAST ABUT. SECTION
HIGHWAY 11
HORIZONTAL SCALE 4 0 4 8 m
2 0 2 4 m VERTICAL SCALE

**METRIC**
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 164-98-00

HIGHWAY 11
MATTAWISHKWIA RIVER BRIDGE
SOIL STRATA

SHEET

Golder Associates
**Golder Associates Ltd.**
SUDBURY, ONTARIO, CANADA

## LEGEND

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LICENSED PROFESSIONAL ENGINEER A.J.K. BOM 100075715 NOV 14, 2011 PROVINCE OF ONTARIO

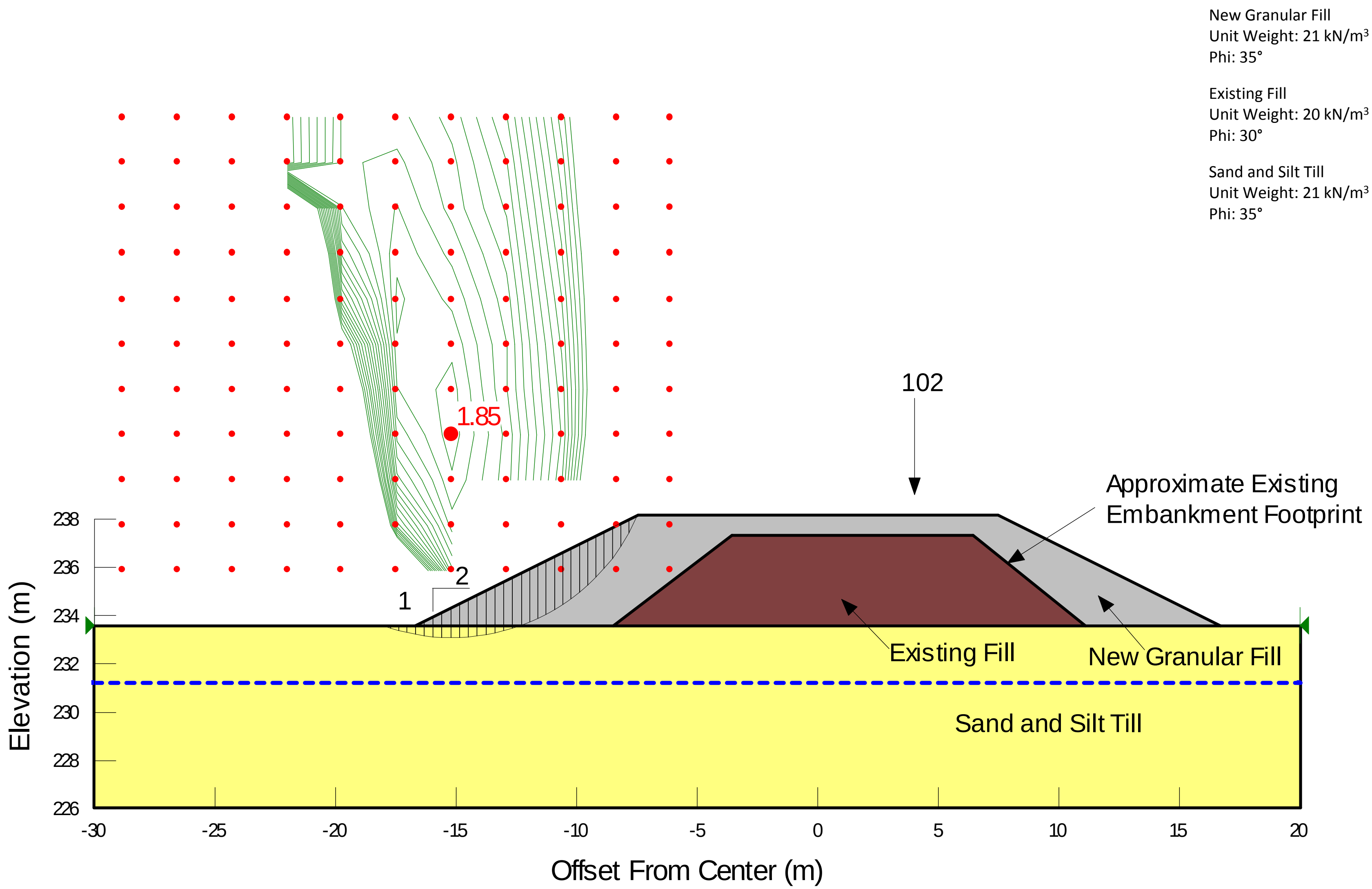
REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN NOV 14, 2011 PROVINCE OF ONTARIO

| NO. | DATE | BY | REVISION |
|---|---|---|---|

Geocres No. 42G-34

| | | | |
|---|---|---|---|
| HWY. 11 | PROJECT NO. 10-1191-0038 | | DIST. |
| SUBM'D. | CHKD. AB | DATE: NOV 2011 | SITE: 39W-33 |
| DRAWN: JJL | CHKD. SEMC | APPD. FJH | DWG. 2 |

PLOT DATE: November 14, 2011




PROJECT

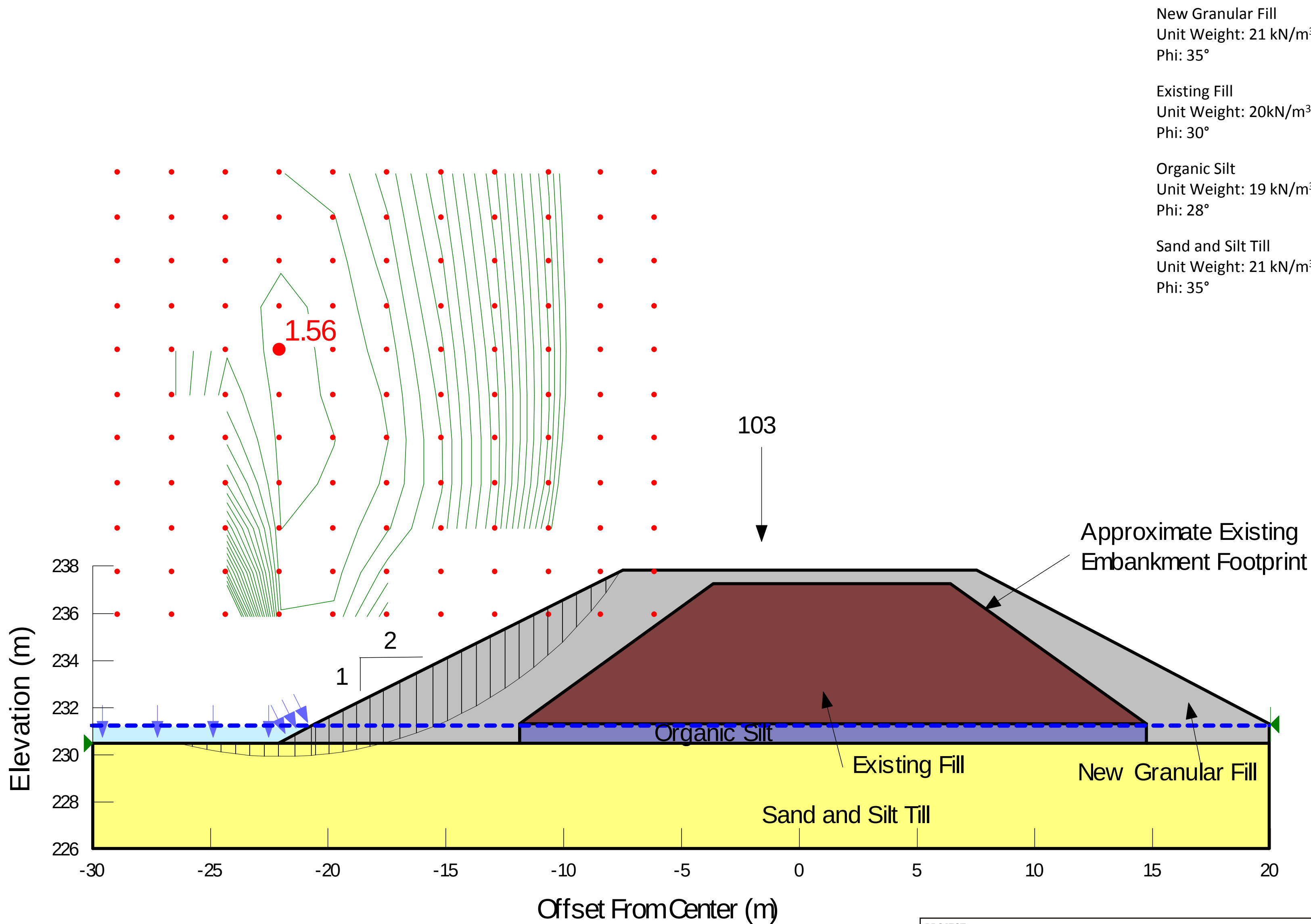
MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11

TITLE

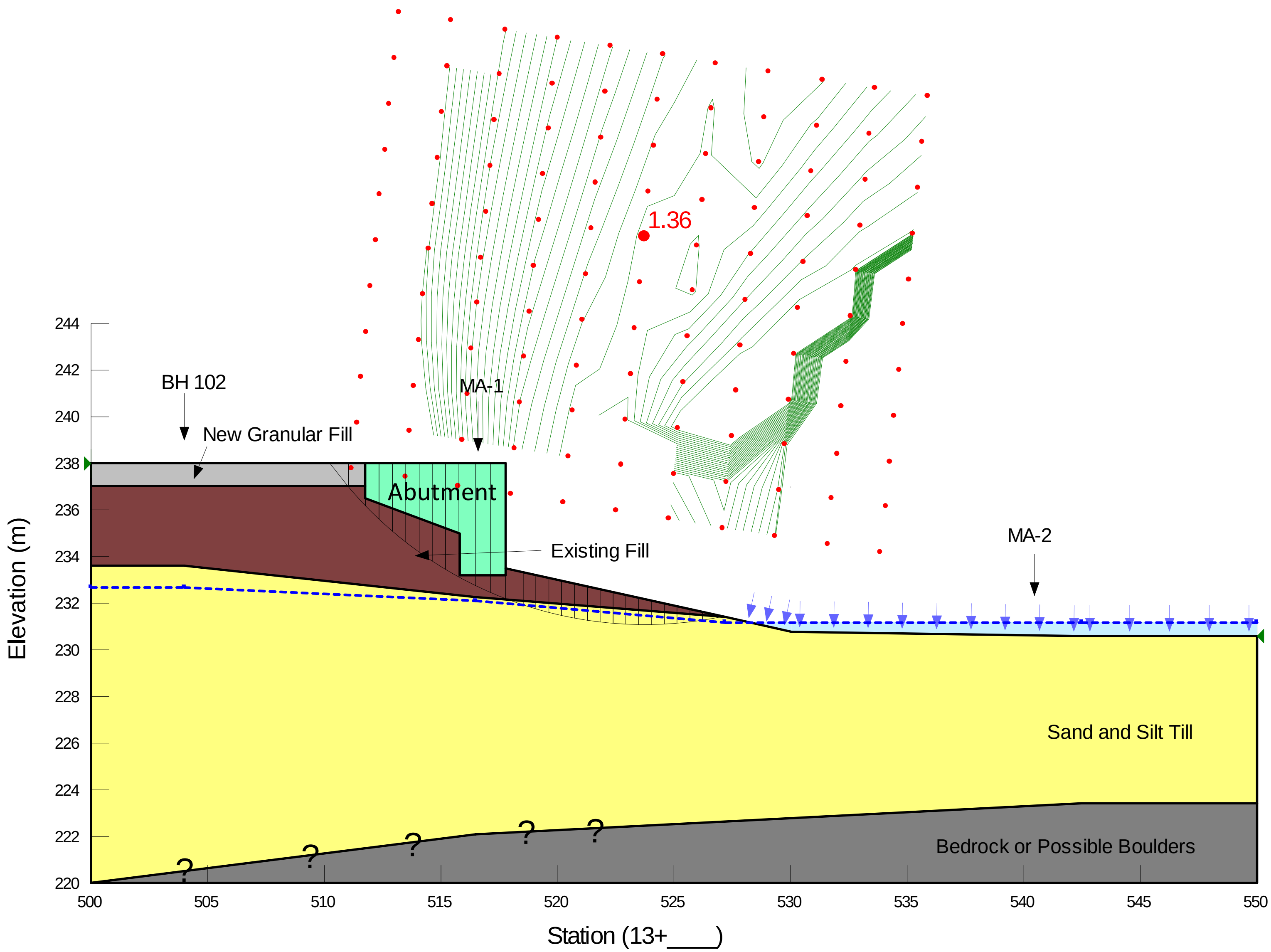
**WEST APPROACH NORTH SIDE SLOPE**

Golder Associates

| | | | | |
|---|---|---|---|---|
| PROJECT No. 10-1191-0038 | | | FILE No. ---- | |
| DESIGN | RH/AB | NOV. 2011 | SCALE AS SHOWN | REV. |
| CADD | -- | | **Figure 1** | |
| CHECK | AB | NOV. 2011 | | |
| REVIEW | FJH | NOV. 2011 | | |




| PROJECT | | | | |
|---|---|---|---|---|
| MATTAWISHKWIA RIVER BRIDGE<br>HIGHWAY 11 | | | | |
| TITLE | | | | |
| **EAST APPROACH NORTH SIDE SLOPE** | | | | |
| Golder Associates | PROJECT No. 10-1191-0038 | | FILE No. ---- | |
| | DESIGN | RH/AB | NOV. 2011 | SCALE AS SHOWN | REV. |
| | CADD | -- | | | |
| | CHECK | AB | NOV. 2011 | **Figure 2** | |
| | REVIEW | FJH | NOV. 2011 | | |




New Granular Fill
Unit Weight: 21 kN/m³
Phi: 35°

Existing Fill
Unit Weight: 20 kN/m³
Phi: 30°

Sand and Silt Till
Unit Weight: 21 kN/m³
Phi: 35°

PROJECT
MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11

TITLE

**WEST APPROACH FRONT SLOPE**

Golder Associates

| | | | | |
|---|---|---|---|---|
| PROJECT No. 10-1191-0038 | | | FILE No. ---- | |
| DESIGN | RH/AB | NOV. 2011 | SCALE AS SHOWN | REV. |
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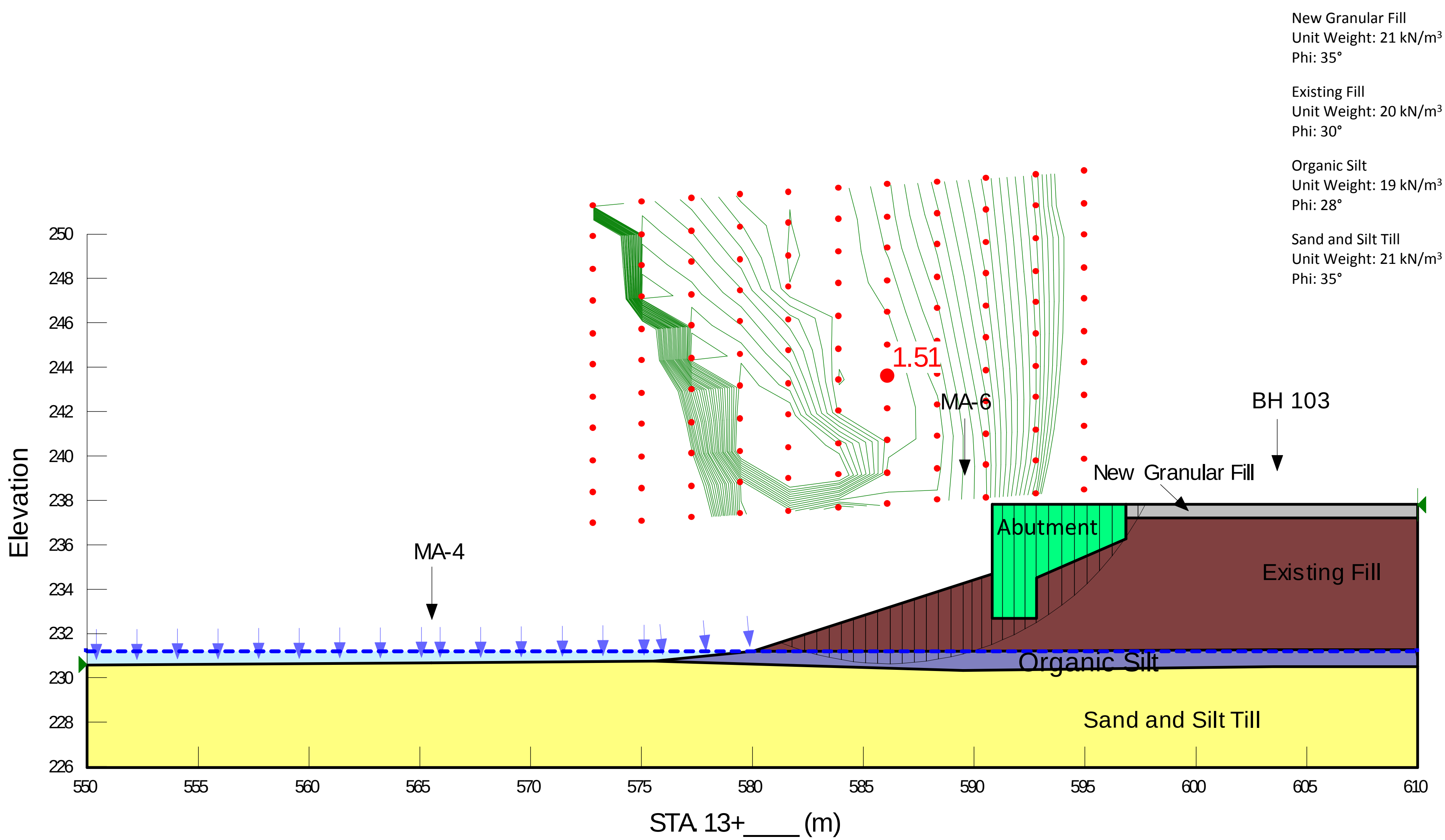
| CHECK | AB | NOV. 2011 | **Figure 3** | |
| REVIEW | FJH | NOV. 2011 | | |




PROJECT

MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11

TITLE

**EAST APPROACH FRONT SLOPE**

Golder Associates

| | | | | |
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| PROJECT No. 10-1191-0038 | | | FILE No. ---- | |
| DESIGN | RH/AB | NOV. 2011 | SCALE AS SHOWN | REV. |
| CADD | -- | | | |
| CHECK | AB | NOV. 2011 | **Figure 4** | |
| REVIEW | FJH | NOV. 2011 | | |

# APPENDIX A

## Record of Boreholes and Drillholes

# LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

## 1. GENERAL

| | |
|---|---|
| $\pi$ | 3.1416 |
| in x, | natural logarithm of x |
| $\log_{10}$ | x or log x, logarithm of x to base 10 |
| g | acceleration due to gravity |
| t | time |
| FoS | Factor of Safety |
| V | volume |
| W | weight |

## II. STRESS AND STRAIN

| | |
|---|---|
| $\gamma$ | shear strain |
| $\Delta$ | change in, e.g. stress: $\Delta\sigma$ |
| $\varepsilon$ | linear strain |
| $\varepsilon_v$ | volumetric strain |
| $\eta$ | coefficient of viscosity |
| $\nu$ | Poisson's ratio |
| $\sigma$ | total stress |
| $\sigma'$ | effective stress ($\sigma' = \sigma$-u) |
| $\sigma_{vo}$ | initial effective overburden stress |
| $\sigma_1, \sigma_2, \sigma_3$ | principal stress (major, intermediate, minor) |
| $\sigma_{oct}$ | mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| $\tau$ | shear stress |
| u | porewater pressure |
| E | modulus of deformation |
| G | shear modulus of deformation |
| K | bulk modulus of compressibility |

## III. SOIL PROPERTIES

### (a) Index Properties

| | |
|---|---|
| $\rho(\gamma)$ | bulk density (bulk unit weight*) |
| $\rho_d(\gamma_d)$ | dry density (dry unit weight) |
| $\rho_w(\gamma_w)$ | density (unit weight) of water |
| $\rho_s(\gamma_s)$ | density (unit weight) of solid particles |
| $\gamma'$ | unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$) |
| $D_R$ | relative density (specific gravity) of solid particles ($D_R = \rho_s/\rho_w$) (formerly $G_s$) |
| e | void ratio |
| n | porosity |
| S | degree of saturation |

* Density symbol is $\rho$. Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity).

### (a) Index Properties (continued)

| | |
|---|---|
| w | water content |
| $w_l$ | liquid limit |
| $w_p$ | plastic limit |
| $I_p$ | plasticity index – $(w_l - w_p)$ |
| $w_s$ | shrinkage limit |
| $I_L$ | liquidity index = $(w - w_p)/I_p$ |
| $I_c$ | consistency index = $(w_l - w)/I_p$ |
| $e_{max}$ | void ratio in loosest state |
| $e_{min}$ | void ratio in densest state |
| $I_D$ | density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density) |

### (b) Hydraulic Properties

| | |
|---|---|
| h | hydraulic head or potential |
| q | rate of flow |
| v | velocity of flow |
| i | hydraulic gradient |
| k | hydraulic conductivity (coefficient of permeability) |
| j | seepage force per unit volume |

### (c) Consolidation (one-dimensional)

| | |
|---|---|
| $C_c$ | compression index (normally consolidated range) |
| $C_r$ | recompression index (over-consolidated range) |
| $C_s$ | swelling index |
| $C_a$ | coefficient of secondary consolidation |
| $m_v$ | coefficient of volume change |
| $c_v$ | coefficient of consolidation |
| $T_v$ | time factor (vertical direction) |
| U | degree of consolidation |
| $\sigma'_p$ | pre-consolidation pressure |
| OCR | over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$ |

### (d) Shear Strength

| | |
|---|---|
| $\tau_p, \tau_r$ | peak and residual shear strength |
| $\phi'$ | effective angle of internal friction |
| $\delta$ | angle of interface friction |
| $\mu$ | coefficient of friction = tan $\delta$ |
| c′ | effective cohesion |
| $c_u, s_u$ | undrained shear strength ($\phi = 0$ analysis) |
| p | mean total stress $(\sigma_1 + \sigma_3)/2$ |
| p′ | mean effective stress $(\sigma'_1 + \sigma'_3)/2$ |
| q | $(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$ |
| $q_u$ | compressive strength $(\sigma_1 + \sigma_3)$ |
| $S_t$ | sensitivity |

**Notes:**
1. $\tau = c' + \sigma' \tan \phi'$
2. Shear strength = (Compressive strength)/2

# LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

## I. SAMPLE TYPE

| | |
|---|---|
| AS | Auger sample |
| BS | Block sample |
| CS | Chunk sample |
| SS | Split-spoon |
| DS | Denison type sample |
| FS | Foil sample |
| RC | Rock core |
| SC | Soil core |
| ST | Slotted tube |
| TO | Thin-walled, open |
| TP | Thin-walled, piston |
| WS | Wash sample |

## II. PENETRATION RESISTANCE

**Standard Penetration Resistance (SPT), N:**
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

**Dynamic Cone Penetration Resistance; $N_d$:**
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

**PH:** Sampler advanced by hydraulic pressure
**PM:** Sampler advanced by manual pressure
**WH:** Sampler advanced by static weight of hammer
**WR:** Sampler advanced by weight of sampler and rod

**Piezo-Cone Penetration Test (CPT)**
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 $cm^2$ pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ($Q_t$), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

## III. SOIL DESCRIPTION

**(a) Cohesionless Soils**

| **Density Index Relative Density** | **N Blows/300 mm or Blows/ft** |
|---|---|
| Very loose | 0 to 4 |
| Loose | 4 to 10 |
| Compact | 10 to 30 |
| Dense | 30 to 50 |
| Very dense | over 50 |

**(b) Cohesive Soils**

| **Consistency** | **$c_u$, $s_u$ kPa** | **psf** |
|---|---|---|
| Very soft | 0 to 12 | 0 to 250 |
| Soft | 12 to 25 | 250 to 500 |
| Firm | 25 to 50 | 500 to 1,000 |
| Stiff | 50 to 100 | 1,000 to 2,000 |
| Very stiff | 100 to 200 | 2,000 to 4,000 |
| Hard | over 200 | over 4,000 |

## IV. SOIL TESTS

| | |
|---|---|
| w | water content |
| $w_p$ | plastic limit |
| $w_l$ | liquid limit |
| C | consolidation (oedometer) test |
| CHEM | chemical analysis (refer to text) |
| CID | consolidated isotropically drained triaxial test[1] |
| CIU | consolidated isotropically undrained triaxial test with porewater pressure measurement[1] |
| $D_R$ | relative density (specific gravity, $G_s$) |
| DS | direct shear test |
| M | sieve analysis for particle size |
| MH | combined sieve and hydrometer (H) analysis |
| MPC | Modified Proctor compaction test |
| SPC | Standard Proctor compaction test |
| OC | organic content test |
| $SO_4$ | concentration of water-soluble sulphates |
| UC | unconfined compression test |
| UU | unconsolidated undrained triaxial test |
| V | field vane (LV-laboratory vane test) |
| $\gamma$ | unit weight |

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

## V. MINOR SOIL CONSTITUENTS

| Percent by Weight | Modifier | Example |
|---|---|---|
| 0 to 5 | Trace | Trace sand |
| 5 to 12 | Trace to Some (or Little) | Trace to some sand |
| 12 to 20 | Some | Some sand |
| 20 to 30 | (ey) or (y) | Sandy |
| over 30 | And (cohesionless) or | Sand and Gravel |
| | With (cohesive) | Silty Clay with sand / Clayey Silt with sand |

# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

## WEATHERING STATE

**Fresh:** no visible sign of weathering

**Faintly weathered:** weathering limited to the surface of Major discontinuities

**Slightly weathered:** penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

**Moderately weathered:** weathering extends throughout the rock mass but the rock material is not friable.

**Highly weathered:** weathering extends throughout rock Mass and the rock material is partly friable.

**Completely weathered:** rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

## BEDDING THICKNESS

| Description | Bedding Plane Spacing |
|---|---|
| Very thickly bedded | > 2 m |
| Thickly bedded | 0.6 m to 2 m |
| Medium bedded | 0.2 m to 0.6 m |
| Thinly bedded | 60 mm to 0.2 m |
| Very thinly bedded | 20 mm to 60 mm |
| Laminated | 6 mm to 20 mm |
| Thinly laminated | < 6 mm |

## JOINT OR FOLIATION SPACING

| Description | Spacing |
|---|---|
| Very wide | > 3 m |
| Wide | 1 – 3 m |
| Moderately close | 0.3 – 1 m |
| Close | 50 – 300 mm |
| Very close | < 50 mm |

## GRAIN SIZE

| Terms | Size* |
|---|---|
| Very Coarse Grained | > 60 mm |
| Coarse Grained | 2 – 60 mm |
| Medium Grained | 60 microns – 2 mm |
| Fine Grained | 2 – 60 microns |
| Very Fine Grained | < 2 microns |

* Note: Grains > 60 microns diameter are visible to the naked eye.

## CORE CONDITION

**Total Core Recovery (TCR)**

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

**Solid Core Recovery (SCR)**

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

**Rock Quality Designation (RQD)**

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

## DISCONTINUITY DATA

**Fracture Index**

A count of the number of discontinuities (physical separation) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

**Dip with Respect to (W.R.T.) Core Axis**

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

**Description and Notes**

An abbreviated description of the discontinuities, whether naturally occurring separation such as fractures, bedding planes and foliation planes or mechanically induced fractures caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

**Abbreviations**

| | | | |
|---|---|---|---|
| B | - Bedding | $\perp$ | - Perpendicular To |
| FO | - Foliation / Schistosity | $\parallel$ | - Parallel To |
| CL | - Cleavage | P | - Polished |
| SH | - Shear Plane / Zone | K | - Slickensided |
| VN | - Vein | SM | - Smooth |
| F | - Fault | R | - Rough |
| CO | - Contact | ST | - Stepped |
| J | - Joint | PL | - Planar |
| FR | - Fracture | U | - Undulating |
| MF | - Mechanical Fracture | C | - Curved |

Golder Associates



# RECORD OF BOREHOLE No MA-1

 **METRIC**

PROJECT 10-1191-0038
W.P. 164-98-00
LOCATION N 5505342.1; E 331222.6
ORIGINATED BY NG
DIST HWY 11
BOREHOLE TYPE HW/HQ Casing/Coring with Wash Boring, Tri-Cone
COMPILED BY PL
DATUM Geodetic
DATE March 16, 2011
CHECKED BY AB

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 237.3 | BRIDGE DECK | | | | | |
| 0.0 | Asphalt (115mm) | | | | | |
| 236.8 | Concrete (370mm) | | | | | |
| 0.5 | Void | | | | | |
| 233.6 3.7 | Silty clay, some sand, trace gravel, trace organics (FILL) Hard Brown to black Moist | 1 | SS | 44 | | |
| 232.3 5.0 | SAND and SILT, trace to some clay with gravel, cobbles and boulders (TILL) Compact to very dense Grey Wet | 2a 2b | SS | 25 | ∇ | |
| | | 3 | SS | 26 | | |
| | | 4 | SS | 43 | | 3 15 70 12 |
| | Slow casing advance below about Elev. 230.2 m. | 5 | SS | 85 | | |
| | Advanced borehole using tri-cone below Elev. 229.2 m. | 6 | SS | 29/0.15 | | |
| | | 7 | SS | 53 | | 17 51 28 4 |
| | Increasing frequency of hard drilling/grinding of tricone below Elev. 228.0 m. | | | | | |
| | Hammer bouncing immediately when attempting to obtain Samples 8 to 10. | 8 | SS | - | | |
| | Cored through three boulders between Elev. 226.8 m and 225.4 m. | | | | | |
| | Cored through several boulders below Elev. 225.3 m. | 9 | SS | - | | |
| | | 10 | SS | - | | |

ELEVATION SCALE: 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT $W_P$ — NATURAL MOISTURE CONTENT W — LIQUID LIMIT $W_L$
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³

Continued Next Page

+3, ×3: Numbers refer to Sensitivity    O 3% STRAIN AT FAILURE

SUD-MTO 001 1011910038 BH LOGS.GPJ GAL-MISS.GDT 09/11/11 DATA INPUT:

Golder Associates

# RECORD OF BOREHOLE No MA-1

 **METRIC**

PROJECT 10-1191-0038
W.P. 164-98-00
LOCATION N 5505342.1; E 331222.6
ORIGINATED BY NG
DIST HWY 11
BOREHOLE TYPE HW/HQ Casing/Coring with Wash Boring, Tri-Cone
COMPILED BY PL
DATUM Geodetic
DATE March 16, 2011
CHECKED BY AB

| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) | PLASTIC LIMIT $w_P$ / NATURAL MOISTURE CONTENT $w$ / LIQUID LIMIT $w_L$ — WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | | | | | | | |
| 222.1<br>15.2 | Sand and gravel obtained from Sample 11 of black, grey, white and pink colours.<br>END OF BOREHOLE | | 11 | SS | 60/0.15 | | | | ○ | | 4 | 80 | 13 | 3 |

Note:

1. Water level at a depth of 1.6 m below ground surface (Elev. 232.0 m) upon completion of drilling.

+³, ×³: Numbers refer to Sensitivity    ○³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No MA-2

 **METRIC**

PROJECT 10-1191-0038
W.P. 164-98-00
DIST HWY 11
DATUM Geodetic
LOCATION N 5505343.4; E 331249.6
BOREHOLE TYPE NW/NQ Casing/Coring with Wash Boring
DATE March 18 to 21, 2011
ORIGINATED BY NG
COMPILED BY PL
CHECKED BY AB

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 231.2 | ICE SURFACE | | | | | | |
| 0.0 | ICE/WATER | | | | 231 | | |
| 230.6 / 0.6 | SAND and SILT, trace to some clay with gravel, cobbles and boulders (TILL) Dense to very dense Grey Wet | 1 | SS | 48 | 230 | o | 11 28 49 12 |
| | | 2 | SS | 94 | 229 | | |
| | | 3 | SS | 69 | | o | 18 43 34 5 |
| | Slow casing advance below Elev. 228.5 m. Gravel pieces jamming in core barrel at Elev. 227.8 m. Cobble (200 mm) encountered at Elev. 227.7 m. | 4 | SS | 160/0.15 | 228 | | |
| | | 5 | RC | - | | | |
| | | 6 | RC | - | | | |
| | Difficult casing advance below Elev. 227.5 m. | 7 | SS | 37 | 227 | o | 57 37 5 1 |
| | Broken rock in top of spoon at Sample 8. | 8 | SS | 78 | 226 | | |
| | | 9 | SS | 70 | 225 | o | 9 63 24 4 |
| | Metasediment boulder (600 mm) encountred at Elev. 224.4 m. Gravel consisting of metasediment, quartz and siltstone encountred at Elev. 223.8 m. Metasediment cobble encountred at Elev. 223.5 m. | 10 | SS | 20/0.1 | 224 | | |
| | | 11 | RC | REC 100% | | | |
| 223.4 / 7.8 | METASEDIMENT (POSSIBLE BEDROCK or BOULDER) Rock cored from 7.8 m depth to 9.9 m depth. For coring details see Record of Drillhole MA-2. | 1 | RC | REC 100% | 223 | | RQD = 100% |
| | | 2 | RC | REC 100% | 222 | | RQD = 97% |
| 221.3 / 9.9 | END OF BOREHOLE | | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED
PLASTIC LIMIT $w_P$, NATURAL MOISTURE CONTENT $w$, LIQUID LIMIT $w_L$
UNIT WEIGHT $\gamma$ kN/m$^3$

$+^3, \times^3$: Numbers refer to Sensitivity    $O^{3\%}$ STRAIN AT FAILURE

# RECORD OF DRILLHOLE: MA-2

PROJECT: 10-1191-0038
LOCATION: N 5505343.4 ;E 331249.6
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 18 to 21, 2011
DRILL RIG: D-25
DRILLING CONTRACTOR: Walker Drilling

SHEET 1 OF 1
DATUM: Geodetic

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate; BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage; PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular; PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break; BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. / DEPTH (m) | RUN No. | DIP w.r.t. CORE AXIS | TYPE AND SURFACE DESCRIPTION | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|---|
| | | REFER TO PREVIOUS PAGE | 223.4 | | | | |
| 8 | March 21, 2011 NQ Coring | Metasediment with white quartz veins (about 1 mm to 50 mm thick)<br>Fine grained<br>Fresh<br>Grey | 7.8 | 1 | | | |
| 9 | | | | 2 | • | JIR | |
| | | | | | • • • | JIR<br>JIR<br>JIR | |
| | | | 221.3 | | • | JIR | |
| 10 | | END OF DRILLHOLE | 9.9 | | | | |

Column headers also include: SYMBOLIC LOG; COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.25 METRES; DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn); HYDRAULIC CONDUCTIVITY k, cm/s; Diametral Point Load Index (MPa); RMC -Q' AVG.

Depth scale: 11, 12, 13, 14, 15, 16, 17

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: NG
CHECKED: AB

SUD-RCK 1011910038 BH LOGS.GPJ GAL-MISS.GDT 09/11/11 DATA INPUT:

Golder Associates

# RECORD OF BOREHOLE No MA-3

 **METRIC**

PROJECT 10-1191-0038
W.P. 164-98-00
LOCATION N 5505330.5; E 331239.8
ORIGINATED BY NG
DIST HWY 11
BOREHOLE TYPE NW/NQ Casing/Coring with Wash Boring
COMPILED BY PL
DATUM Geodetic
DATE March 21 and 22, 2011
CHECKED BY AB

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 231.1 | ICE SURFACE | | | | | | |
| 0.0 | ICE/WATER | | | | 231 | | |
| 230.5 / 0.6 | SAND and SILT, trace to some clay with gravel, cobbles and boulders (TILL) Dense to very dense Grey Wet | 1 | SS | 46 | 230 | o | 12 41 39 8 |
| | | 2 | SS | 134 | 229 | | |
| | | 3 | SS | 94 | | o | 29 46 21 4 |
| | Slow casing advance below Elev. 228.1 m. | 4 | SS | 70 | 228 | | |
| | | 5 | SS | 60 | 227 | o | 29 50 17 4 |
| | | 6 | SS | 100/0.1 | | | |
| 226.0 | Quartz boulder (400 mm) over metasediment cobble (100 mm) at Elev. 226.5 m. | 7 | RC | REC 100% | | | |
| 5.1 | METASEDIMENT (POSSIBLE BEDROCK or BOULDER) Rock cored from 5.1 m depth to 6.6 m depth. For coring details see Record of Drillhole MA-3. | 1 | RC | REC 100% | 226 / 225 | | RQD = 97% |
| 224.5 / 6.6 | END OF BOREHOLE | | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

+3, ×3: Numbers refer to Sensitivity
O 3% STRAIN AT FAILURE

SUD-MTO 001 1011910038 BH LOGS.GPJ GAL-MISS.GDT 09/11/11 DATA INPUT:

PROJECT: 10-1191-0038

# RECORD OF DRILLHOLE: MA-3

SHEET 1 OF 1

LOCATION: N 5505330.5 ;E 331239.8

DRILLING DATE: March 21 and 22, 2011

DATUM: Geodetic

DRILL RIG: D-25

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. / DEPTH (m) | RUN No. | DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|---|
| | | REFER TO PREVIOUS PAGE | | 226.0 | | | |
| 6 | March 22, 2011 NQ Coring | Metasediment with white quartz veins (5 mm to 300 mm thick)<br>Fine grained<br>Fresh<br>Grey | | 5.1 | 1 | JPLR<br>JPLR<br>JPLR<br>JPLR<br>JPLR<br>JPLR | |
| | | | | 224.5<br>6.6 | | | |
| | | END OF DRILLHOLE | | | | | |

Depth scale: 7, 8, 9, 10, 11, 12, 13, 14, 15

Column headings: COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3 METRES; B Angle; DIP w.r.t. CORE AXIS; Jr; Ja; Jn; HYDRAULIC CONDUCTIVITY k, cm/s ($10^{-6}$, $10^{-5}$, $10^{-4}$, $10^{-3}$); Diametral Point Load Index (MPa); RMC -Q' AVG.

SUD-RCK 1011910038 BH LOGS.GPJ GAL-MISS.GDT 09/11/11 DATA INPUT:

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: NG

CHECKED: AB

Golder Associates

PROJECT 10-1191-0038

# RECORD OF BOREHOLE No MA-4

 **METRIC**

W.P. 164-98-00 LOCATION N 5505322.6; E 331268.1 ORIGINATED BY NG

DIST HWY 11 BOREHOLE TYPE NW/NQ Casing/Coring with Wash Boring COMPILED BY PL

DATUM Geodetic DATE March 23 and 24, 2011 CHECKED BY AB

| ELEV DEPTH (m) | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 231.3 | ICE SURFACE | | | | | | |
| 0.0 | ICE/WATER | | | | 231 | | |
| 230.7<br>0.6 | SAND and SILT, trace to some clay with gravel, cobbles and boulders (TILL)<br>Very dense<br>Grey<br>Wet | 1 | SS | 64 | 230 | | |
| | | 2 | SS | - | | | |
| | Several 10 mm to 75 mm gravel pieces recovered in core barrel at Elev. 229.6 m. | 3 | RC | - | 229 | | |
| | | 4 | SS | 51 | | o | 37 46 (17) |
| | Hammer bouncing at Samples 5 and 7. | 5 | SS | 20/0.1 | 228 | | |
| | Metasediment boulder (300 mm) encountered at Elev. 227.7 m. | 6 | RC | - | | | |
| | | 7 | SS | 100/0.15 | | o | 31 59 (10) |
| | Metasediment boulder (750 mm) with quartz veins encountered at Elev. 227.2 m; sand on core barrel when boulder retrieved. | 8 | RC | REC 100% | 227 | | |
| | | | | | 226 | | |
| | Gravel and cobbles (quartz and metasediment) recovered in core barrel at Elev. 225.8 m. | 9 | RC | REC 100% | | | |
| 225.4<br>5.9 | METASEDIMENT (POSSIBLE BEDROCK or BOULDER)<br>Rock cored from 5.9 m depth to 7.0 m depth. | 1 | RC | REC 100% | 225 | | RQD = 100% |
| 224.3<br>7.0 | For coring details see Record of Drillhole MA-4.<br>END OF BOREHOLE | | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

SUD-MTO 001 1011910038 BH LOGS.GPJ GAL-MISS.GDT 09/11/11 DATA INPUT:

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

PROJECT: 10-1191-0038

# RECORD OF DRILLHOLE: MA-4

SHEET 1 OF 1

LOCATION: N 5505322.6 ;E 331268.1

DRILLING DATE: March 23 and 24, 2011

DATUM: Geodetic

DRILL RIG: D-25

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate; BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage; PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular; PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break; BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION | NOTES WATER LEVELS INSTRUMENTATION |
|---|---|---|---|---|---|---|---|
| | | REFER TO PREVIOUS PAGE | | 225.4 | | | |
| 6 | March 24, 2011 NQ Coring | Metasediment with white quartz veins (1 mm to 5 mm thick)<br>Fine grained<br>Fresh<br>Grey | | 5.9 | 1 | JPLR<br>JPLR<br>JPLR<br>JIR | |
| 7 | | | | 224.3 | | | |
| | | END OF DRILLHOLE | | 7.0 | | | |
| 8 | | | | | | | |
| 9 | | | | | | | |
| 10 | | | | | | | |
| 11 | | | | | | | |
| 12 | | | | | | | |
| 13 | | | | | | | |
| 14 | | | | | | | |
| 15 | | | | | | | |

Column headings also include: COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3 METRES; B Angle; DIP w.r.t. CORE AXIS; Jr; Ja; Jn; HYDRAULIC CONDUCTIVITY k, cm/s ($10^{-6}$, $10^{-5}$, $10^{-4}$, $10^{-3}$); Diametral Point Load Index (MPa); RMC -Q' AVG.

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: NG

CHECKED: AB

SUD-RCK 1011910038 BH LOGS.GPJ GAL-MISS.GDT 09/11/11 DATA INPUT:

# RECORD OF BOREHOLE No MA-5

 **METRIC**

PROJECT 10-1191-0038
W.P. 164-98-00
LOCATION N 5505337.4; E 331279.0
ORIGINATED BY NG
DIST
HWY 11
BOREHOLE TYPE NW/NQ Casing/Coring with Wash Boring
COMPILED BY PL
DATUM Geodetic
DATE March 24, 2011
CHECKED BY AB

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 231.2 | ICE SURFACE | | | | | | |
| 0.0 | ICE/WATER | | | | 231 | | |
| 230.7 / 0.5 | SAND and SILT, trace to some clay with gravel, cobbles and boulders (TILL) Very Dense Grey Wet | | | | 230 | | |
| | | 1 | SS | 2/0.02 | | | |
| | | 2 | SS | 110/0.15 | 229 | | |
| | | 3 | SS | 170/0.15 | 228 | o | 58 36 (6) |
| | | 4 | SS | 30/0.1 | | | |
| | Gravel and cobbles (quartz and metasediment) recovered from core barrel below Elev. 227.2 m. | 5 | RC | REC 100% | 227 | | |
| 226.1 | | 6 | RC | REC 100% | | | |
| 5.1 | END OF BOREHOLE | | | | | | |

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+3, ×3: Numbers refer to Sensitivity    O 3% STRAIN AT FAILURE

Golder Associates

# RECORD OF BOREHOLE No MA-6

 **METRIC**

PROJECT 10-1191-0038
W.P. 164-98-00
LOCATION N 5505319.0; E 331291.6
ORIGINATED BY NG
DIST HWY 11
BOREHOLE TYPE NW/NQ Casing/Coring with Wash Boring
COMPILED BY PL
DATUM Geodetic
DATE March 22 and 23, 2011
CHECKED BY AB

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|
| 235.9 | GROUND SURFACE | | | | |
| 0.0 | Sand and gravel (FILL)<br>Brown<br>Moist | | | | |
| 235.4<br>0.5 | Clayey silt, trace to some sand, some gravel, occasional pockets of organic material (FILL)<br>Firm to stiff<br>Brown<br>Moist | 1 | SS | 13 | |
| | | 2 | SS | 8 | |
| | | 3 | SS | 7 | 12 11 40 38 |
| | | 4 | SS | 8 | |
| | | 5 | SS | 9 | |
| 231.2<br>4.7 | Sandy organic SILT, trace gravel<br>Compact<br>Black<br>Moist | 6 | SS | 19 | |
| 230.3<br>5.6 | SAND and SILT, trace to some clay with gravel, cobbles and boulders (TILL)<br>Very dense<br>Grey<br>Wet | 7 | SS | 134 | 20 50 26 4 |
| | Slow casing advance below Elev. 228.7 m. | 8 | RC | REC 100% | |
| | Metasediment boulder (300 mm) cored at Elev. 228.6 m. | 9 | SS | 127 | |
| | | 10 | SS | 205 | 23 66 (11) |
| 226.3<br>9.6 | END OF BOREHOLE | | | | |

Note:

1. Water level at a depth of 7.1 m below ground surface (Elev. 228.8 m) upon completion of drilling.

2. On March 24, 2011 water level in piezometer at a depth of 5.1 m below existing ground surface (Elev. 230.8 m).

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (235, 234, 233, 232, 231, 230, 229, 228, 227); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT $W_P$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

+3, ×3: Numbers refer to Sensitivity    O 3% STRAIN AT FAILURE

# APPENDIX B

## Record of Boreholes, MTC (1978) and PML (2008)

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION – ENGINEERING MATERIALS OFFICE – SOIL MECHANICS SECTION



# RECORD OF BOREHOLE No 1

W P 236-77-00 LOCATION Sta. 380+90.0, Rt. 12.8' ORIGINATED BY C.T.J.

DIST 16 HWY 11 BOREHOLE TYPE Casing and Wash Boring COMPILED BY C.T.J.

DATUM Geodetic DATE July 30, 1978 CHECKED BY [illegible]

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) | PLASTIC LIMIT $W_p$ – NATURAL MOISTURE CONTENT W – LIQUID LIMIT $W_L$ / WATER CONTENT (%) (10 20 30) | UNIT WEIGHT $\gamma$ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | 'N' VALUES | | | | | | |
| 759.3 | Ground Level | | | | | | | | | | |
| 0.0 | Boulders | | | | | | | | | | |
| 756.3 | Clayey Silt, Some Sand, Firm | | 1 | SS | 5 | | | | | | |
| 3.0 | Sandy Silt, Trace of Gravel and Clay (Glacial Till) | | 2 | SS | 34 | | | | o | | 9 40 45 6 |
| | | | 3 | SS | 50 | | | | o | | |
| 750.8 | Dense to Very Dense | | 4 | SS | 143 | | | | | | |
| 8.5 | End of Borehole | | | | | | 750 | | | | |



+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION



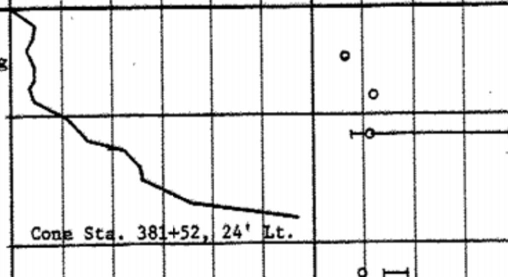
# RECORD OF BOREHOLE No 2

W P 236-77-00 LOCATION Sta. 381+47.0, Lt. 10.5' ORIGINATED BY C.T.J.

DIST 16 HWY 11 BOREHOLE TYPE Continuous Flight Auger COMPILED BY C.T.J.

DATUM Geodetic DATE August 1, 1978 CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | PLASTIC LIMIT $W_p$ / NATURAL MOISTURE CONTENT W / LIQUID LIMIT $W_L$ — WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | 'N' VALUES | | | | | | |
| 778.2 | Pavement Surface | | | | | | | | | | |
| 0.0 | Asphalt, Reinforced Concrete | | | | | | | | | | |
| 1.5 | Fill Material, Sand & Gravel, Traces of Silt & Clay, Occ. Boulders | | 1 | SS | 100/ | 4" Bouncing | | | | | |
| | Loose | | 2 | SS | 10 | | 770 | | | | |
| | Silty Clay, Traces of Sand and Gravel, Occ. Layers of Organics up to ¼" | | 3 | SS | 11 | | | | | | |
| | | | 4 | SS | 11 | ▼ | | | | | 0 17 49 34 |
| 762.7 | Stiff | | 5 | SS | 41 | | | | | | 0 6 91 3 |
| 15.5 | Silt, Tr. of Sand & Clay | | | | | | 760 | Cone Sta. 381+52, 24' Lt. | | | |
| | Sandy Silt, Trace of Gravel and Clay (Glacial Till) Dense to Very Dense | | 6 | SS | 103/ | 6" | | | | | |
| | | | | | | | | | | | 2 34 56 8 |
| 752.2 | | | 7 | SS | 110/ | 6" | | | | | |
| 26.0 | End of Borehole | | | | | | | | | | |



+3, ×5 : Numbers refer to Sensitivity

20 15 ◇ 5 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION–ENGINEERING MATERIALS OFFICE–SOIL MECHANICS SECTION



# RECORD OF BOREHOLE No 3

W P 236-77-00 LOCATION Sta. 379+01.0, Lt. 9' ORIGINATED BY C.T.J.

DIST 16 HWY 11 BOREHOLE TYPE Continuous Flight Auger COMPILED BY C.T.J.

DATUM Geodetic DATE August 1, 1978 CHECKED BY [illegible]

| SOIL PROFILE | | SAMPLES | | | | | | |
|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
| 778.3 | Pavement Surface | | | | | | | |
| .3 | Asphaltic Reinforced Concrete | | | | | | | |
| 1.1 | Void | | | | | | | |
| 2.6 | Reinforced Concrete | | | | | | | |
| 3.6 | Fill Material, Sand & Gravel, Traces of Silt & Clay, Occasional Cobbles & Wood Inclusions Very Loose to Compact | 1 | SS | 4 | | | | |
| | | 2 | SS | 7 | | 770 | Cone Sta. 379+01, 20' Lt. 130/10" | 41 52 6 1 |
| | | 3 | SS | 19 | | | | |
| | Silty Clay, Trace of Sand & Gravel Very Stiff | 4 | SS | 22 | | | | 5 11 54 30 |
| 762.7 | | 5 | SS | 23 | | | | |
| 15.6 | Sandy Silt, Trace of Gravel and Clay, Occ. Cobbles, Very Dense Boulder at elev. 758 (Glacial Till) | | | | ▼ | 760 | | |
| | | 6 | SS | 53 | | | | |
| | Sandy Gravel | 7 | SS | 100/ 5" | | | | 51 29 17 3 |
| 748.2 | | 8 | SS | 100/ 14" | | 750 | | 5 49 42 4 |
| 30.1 | End of Borehole | | | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT $W_p$ — NATURAL MOISTURE CONTENT $W$ — LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30

UNIT WEIGHT $\gamma$

$+^3$, $\times^5$ : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

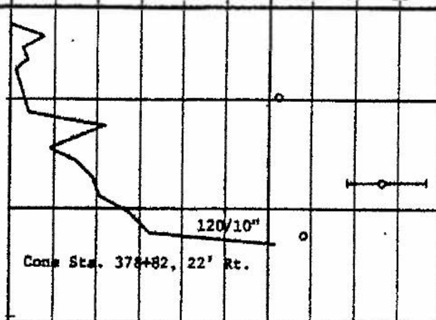
# RECORD OF BOREHOLE No 4



W P 236-77-00 LOCATION Sta. 378+88.0, Rt. 10.5' ORIGINATED BY C.T.J.
DIST 16 HWY 11 BOREHOLE TYPE Continuous Flight Auger COMPILED BY C.T.J.
DATUM Geodetic DATE August 1 & 2, 1978 CHECKED BY [illegible]

| SOIL PROFILE: ELEV / DEPTH | DESCRIPTION | STRAT PLOT | SAMPLES: NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | PLASTIC LIMIT $W_p$ — NATURAL MOISTURE CONTENT $W$ — LIQUID LIMIT $W_L$; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT $\gamma$ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 778.3 | Pavement Surface | | | | | | | | | |
| .4 | Asphalt, Reinforced | | | | | * | | | | |
| 1.4 | Void Concrete | | | | | | | | | |
| 2.9 | Fill Material, Sand & Gravel, Traces of Silt & Clay, Occ. Cobbles & Wood Inclusions, Very Loose to Compact | | 1 | SS | 4 | | | | | |
| | | | 2 | SS | 9 | | 770 | | | 45 48 5 2 |
| | | | 3 | SS | 15 | | | | | |
| | Silty Clay, Trace of Sand & Gravel, Occ. Layers of Organics Up to ½" | | 4 | SS | 13 | | | | | |
| 757.8 | Stiff | | | | | | 760 | | | |
| 20.5 | Sandy Silt, Trace of Gravel and Clay Very Dense (Glacial Till) | | 5 | SS | 53 | | | | | 5 43 46 6 |
| | | | 6 | SS | 100/ | 6" | | | | |
| 749.3 / 29.0 | Refusal to Augering Probable Boulders | | | | | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE




*Note: Water Level Not Established

237.2 m
231.0 m
228.4 m

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity    15 ◇ 5 (%) STRAIN AT FAILURE (20, 10)

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

# RECORD OF BOREHOLE No. 5

W P 236-77-00 LOCATION Sta. 381+22.5 Rt. 10.5' ORIGINATED BY C.T.J.
DIST 16 HWY 11 BOREHOLE TYPE Continuous Flight Auger COMPILED BY C.T.J.
DATUM Geodetic DATE August 1 & 2, 1978 CHECKED BY [illegible]

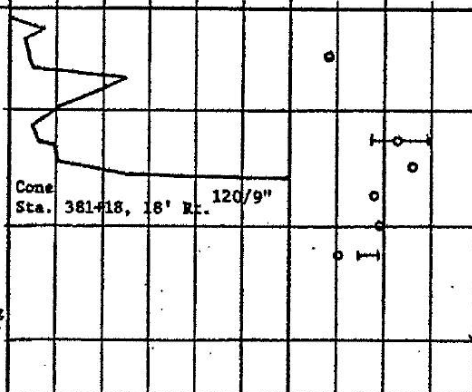
| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 778.3 | Pavement Surface | | | | | | |
| .3 | Asphalt-Reinforced Concrete | | | | * | | |
| 1.2 | Fill Material Sand & Gravel Traces of Silt & Clay Occasional Cobbles Loose | 1 | SS | 9 | | | |
| | | | | | | 770 | |
| | Clayey Silt | 2 | SS | 22 | | | |
| 763.3 | Silty Clay, Trace of Sand Occ. Layer of Organics Up to 1", Very Stiff | 3 | SS | 24 | | | 0 6 56 38 |
| 15.0 | Silt, Traces of Sand & Clay, Dense to Very Dense | 4 | SS | 66 | | | 0 6 91 3 |
| | | 5 | SS | 38 | | 760 | |
| | Sandy Silt, Trace of Gravel and Clay Occasional Cobble Very Dense (Glacial Till) | 6 | SS | 130 | | | |
| | | 7 | SS | 100/5" | Bouncing | | 4 35 54 7 |
| | | | | | | 750 | |
| 745.3 | | | | | | | |
| 33.0 | Refusal to Augering Probable Boulder | | | | | | |
| | *Note: Water Level Not Established | | | | | | |

237.2 m
232.7 m
227.2 m

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH
o UNCONFINED + FIELD VANE
• QUICK TRIAXIAL x LAB VANE

PLASTIC LIMIT $W_p$ NATURAL MOISTURE CONTENT $W$ LIQUID LIMIT $W_L$
WATER CONTENT (%) 10 20 30

UNIT WEIGHT $\gamma$




OFFICE REPORT ON SOIL EXPLORATION

$+^3, \times^5$: Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation
Ontario

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Foundation Design

# RECORD OF BOREHOLE No 102



**METRIC**

| | | | |
|---|---|---|---|
| G.W.P. 154-98-00 | LOCATION Coordinates: 5 505 345.2 N; 331 210.5 E | | ORIGINATED BY F.P. |
| DIST 53 HWY 11 | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | | COMPILED BY A.S. |
| DATUM Geodetic | DATE November 13, 2008 | | CHECKED BY C.N. |

## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 237.3 | Ground Surface |
| 0.0 | 100mm asphaltic concrete over sand, some gravel, some silt (PAVEMENT FILL) |
| 235.8 / 1.5 | Sand, trace gravel; Dense Brown Moist (FILL) |
| | Silt trace clay, trace sand; Loose Grey Moist |
| 233.6 / 3.7 | Silt some clay, trace sand; Compact to dense Brown Moist (TILL) |
| | trace gravel; Very dense Grey |
| 228.1 / 9.2 | End of borehole |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | Natural moisture content (water content %, approx.) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|
| 1 | CS | - | ~3 | 20 70 (10) |
| 2 | SS | 35 | ~3 | |
| 3 | SS | 8 | ~23 | |
| 4 | SS | 32 | ~17 | |
| 5 | SS | 21 | ~18 | |
| 6 | SS | 31 | ~17 | 0 8 81 11 |
| 7 | SS | 50/10cm | ~12 | |
| 8 | SS | 50/8cm | | |
| 9 | SS | 50/8cm | | |

GROUND WATER CONDITIONS: water level ▽* at approx. elevation 232.8

ELEVATION SCALE: 237, 236, 235, 234, 233, 232, 231, 230, 229

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT $w_P$ — NATURAL MOISTURE CONTENT $w$ — LIQUID LIMIT $w_L$; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Samples 7, 8, 9: Sampler bouncing

* 2008 11 13

▽ Water level observed during drilling



+7, ×5 : Numbers refer to Sensitivity

20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

**Peto MacCallum Ltd.**
Foundation Design

# RECORD OF BOREHOLE No 103



**METRIC**

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 325.0 N; 331 307.7 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 12 & 13, 2008 CHECKED BY C.N.

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 237.3 | Ground Surface | | | | | | |
| 0.0 | 100mm asphaltic concrete over sand, some gravel, some silt (PAVEMENT FILL) | 1 | CS | - | 237 | | |
| 235.8 / 1.5 | Sand, trace gravel. Compact to loose, Brown, Moist (FILL) | 2 | SS | 14 | 236 / 235 | o | 14 76 (10) |
| | Silty clay, trace sand, trace gravel, organic inclusions. Stiff, Dark brown | 3 | SS | 8 | 234 | | |
| | | 4 | SS | 7 | 233 | | |
| | | 5 | SS | 6 | | | |
| | | 6 | SS | 7 | 232 | wP–w–wL | 1 9 43 47 |
| | Organic silt | 7 | SS | 12 | 231 | 86 | |
| 230.5 / 6.8 | Sandy silt, trace gravel, cobbles. Very dense, Grey, Moist (TILL) | 8 | SS | 50/5cm | 230 | | |
| | | 9 | SS | 50/3cm | 229 | | |
| | | 10 | SS | 50/8cm | 228 | | |
| 227.7 / 9.6 | Sand with silt with gravel, trace clay. Very dense, Grey, Moist (TILL) | 11 | SS | 95/25cm | | | 22 48 24 6 |
| 227.1 / 10.2 | End of borehole | | | | | | |

Samples 8, 9, 10, 11: Sampler bouncing

* Borehole dry

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100; PLASTIC LIMIT $w_P$, NATURAL MOISTURE CONTENT $w$, LIQUID LIMIT $w_L$; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT



$+^7, \times^5$: Numbers refer to Sensitivity. 20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 203



**METRIC**

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 335.0 N; 331 201.9 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE C.F.H.S.A. + Wash Bore COMPILED BY A.S.

DATUM Geodetic DATE November 16 & 17, 2008 CHECKED BY C.N.

| ELEV DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE / SHEAR STRENGTH kPa | WATER CONTENT (%) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 234.8 0.0 | Ground Surface | | | | * | | | | | |
| 234.6 0.2 | Topsoil | | | | | | | | | |
| 234.2 0.6 | Silty clay organics inclusions Soft (FILL) | 1 | SS | 3 | | | | | | |
| | Silty clay, trace sand Very stiff Brown Moist | 2 | SS | 7 | | 234 | 138 ■ | ○ | | |
| | | 3 | SS | 5 | | 233 | ■ | ○ | | |
| 232.5 2.3 | Silt, trace sand trace gravel, trace clay Compact Brown Moist (TILL) | 4 | SS | 21 | | 232 | | ○ | | 2 6 86 6 |
| | with gravel, some sand some clay Dense to very dense Grey Moist to wet | 5 | SS | 31 | | 231 | | ○ | | |
| | | 6 | SS | 109 | | 230 | | ○ | | 20 17 53 10 |
| | trace clay boulders | 7 | SS | 50/13cm | | | | | | |
| | | 8 | CS | - | | 229 | | | | |
| | | 9 | SS | 50/13cm | | | | | | |
| 227.9 6.9 | End of borehole | 10 | SS | 30/3cm | | 228 | | | | |

Samples 7, 9, 10: Sampler bouncing

* Borehole dry upon completion of drilling

■ Penetrometer test

C.F.H.S.A. - Denotes: Continuous Flight Hollow Stem Augers

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT $w_P$, NATURAL MOISTURE CONTENT $w$, LIQUID LIMIT $w_L$; WATER CONTENT (%) 20 40 60



$+^{7}$, $\times^{5}$ : Numbers refer to Sensitivity

20 / 15 —○— 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

**Peto MacCallum Ltd.**
Foundation Design

## RECORD OF BOREHOLE No 204



**METRIC**

| | | | |
|---|---|---|---|
| G.W.P. 154-98-00 | LOCATION Coordinates: 5 505 308.0 N; 331 290.7 E | | ORIGINATED BY F.P. |
| DIST 53 HWY 11 | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | | COMPILED BY A.S. |
| DATUM Geodetic | DATE November 19, 2008 | | CHECKED BY C.N. |

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 232.5 / 0.0 | Ground Surface | | | | | | |
| 232.3 / 0.2 | Topsoil | | | | | | |
| | Silty sand; Loose to compact; Brown; (FILL) | 1 | SS | 1 | ∇* | 232 | |
| | | 2 | SS | 4 | | 231 | |
| | | 3 | SS | 13 | | | |
| 230.4 / 2.1 | Sand and gravel, with silt trace clay , trace gravel cobbles and boulders; Dense to very dense; Grey; Moist; (TILL) | 4 | SS | 30/5cm | | 230 | |
| | | 5 | SS | 41 | | | 36 36 22 6 |
| | | 6 | SS | 50/10cm | | 229 | |
| | | 7 | SS | 30/10cm | | 228 | 30 46 20 4 |
| 227.8 / 4.7 | End of borehole | | | | | | |

Samples 4, 6, 7: Sampler bouncing

* 2008 11 19

∇ Water level observed during drilling

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100; PLASTIC LIMIT $w_P$, NATURAL MOISTURE CONTENT w, LIQUID LIMIT $w_L$; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³



+7 , ×5 : Numbers refer to Sensitivity; 20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 205



METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 307.7 N; 331 299.9 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 19, 2008 CHECKED BY C.N.

| ELEV / DEPTH | DESCRIPTION | SAMPLE NUMBER | SAMPLE TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa | WATER CONTENT (%) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 233.4 / 0.0 | Ground Surface | | | | * | | | | | |
| 233.2 / 0.2 | Topsoil | | | | | | | | | |
| | Cobbles and boulders | 1 | SS | 6 | | 233 | | | | |
| 232.8 / 0.6 | End of borehole<br>Refusal on probable cobbles and boulders ( possible rockfill ) | | | | | | | | | |

* Borehole dry



+7, ×5 : Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

**Peto MacCallum Ltd.**
Foundation Design

# RECORD OF BOREHOLE No 301



**METRIC**

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 350.6 N; 331 224.9 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 14, 2008 CHECKED BY C.N.

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 237.0 | Ground Surface | | | | | |
| 0.0 | Sand and gravel, trace silt. Loose, Brown, Moist (FILL) | 1 | SS | 9 | | |
| | Sand, trace silt | 2 | SS | 6 | | |
| 235.5 / 1.5 | Silty clay, trace sand. Stiff to firm, Brown, Moist | 3 | SS | 9 | | |
| | | 4 | SS | 8 | | |
| 234.0 / 3.0 | Silt, some sand, trace gravel, trace clay, sandy silt layers. Compact, Brown, Wet (TILL) | 5 | SS | 12 | ∇* | |
| | | 6 | SS | 36 | | |
| | Dense, Grey, Moist to wet | 7 | SS | 36 | | 3 13 76 8 |
| | | 8 | SS | 39 | | |
| | Very dense | 9 | SS | 85/23cm | | |
| 230.5 / 6.5 | End of borehole | | | | | |

Sample 9: Sampler bouncing

* 2008 11 14

∇ Water level observed during drilling

■ Penetrometer test

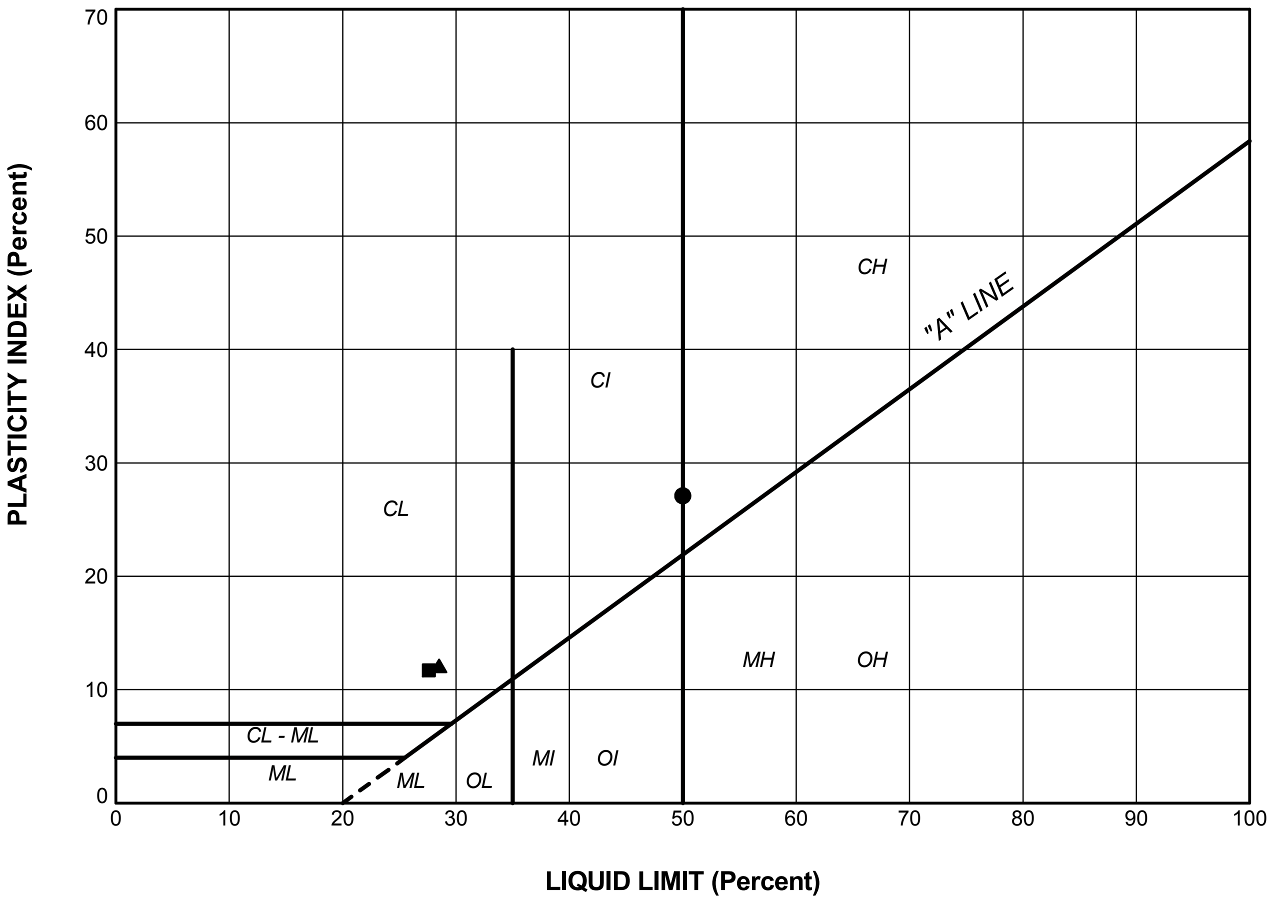


+7, ×5: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

# APPENDIX C

## Laboratory Test Results – Current Investigation




**SOIL TYPE**
C = Clay
M = Silt
O = Organic

**PLASTICITY**
L = Low
I = Intermediate
H = High

## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | LL(%) | PL(%) | PI |
|---|---|---|---|---|---|
| ● | MA-1 | 2a | 50.0 | 22.9 | 27.1 |
| ■ | MA-6 | 2 | 27.6 | 15.9 | 11.7 |
| ▲ | MA-6 | 4 | 28.5 | 16.4 | 12.1 |

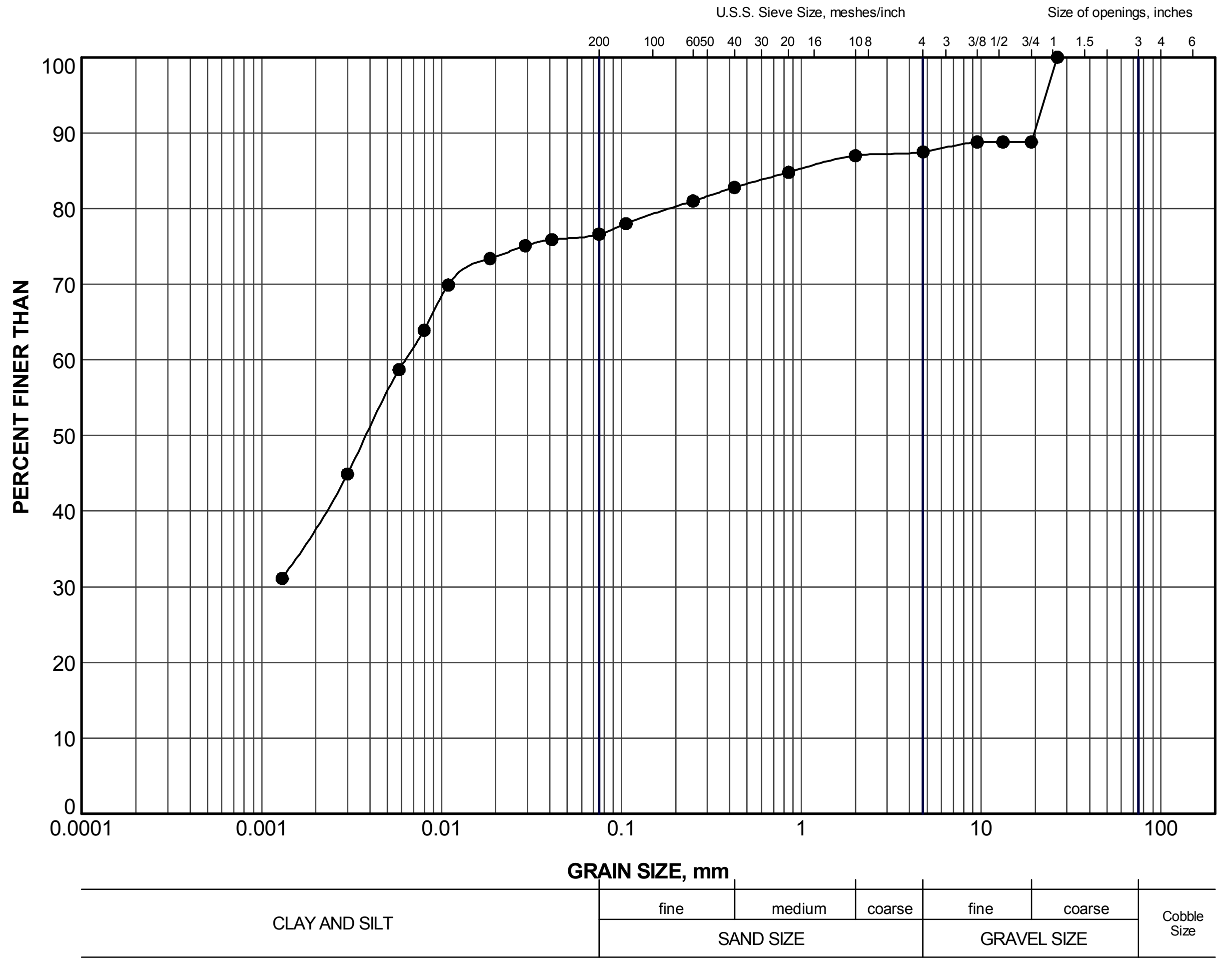
PROJECT
MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11

TITLE
# PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY (FILL)

Golder Associates
SUDBURY, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 10-1191-0038 | | FILE No. 1011910038 BH LOGS.GPJ | |
| DRAWN | JJL | Nov 2011 | SCALE N/A | REV. |
| CHECK | AB | Nov 2011 | | |
| APPR | FJH | Nov 2011 | **FIGURE C1** | |

MTO_PI GLDR_LDN.GDT




**LEGEND**

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | MA-6 | 3 | 233.3 |

PROJECT

MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11

TITLE

# GRAIN SIZE DISTRIBUTION

CLAYEY SILT (FILL)

Golder Associates
SUDBURY, ONTARIO

| | | | | |
|---|---|---|---|---|
| PROJECT No. | | 10-1191-0038 | FILE No. | 1011910038 BH LOGS.GPJ |
| DRAWN | JJL | Nov 2011 | SCALE N/A | REV. |
| CHECK | AB | Nov 2011 | **FIGURE C2** | |
| APPR | FJH | Nov 2011 | | |

LDN_MTO_NEW GLDR_LDN.GDT

MA-2:
Elev. 227.8 m to 227.4 m and
Elev. 224.4 m to 221.3 m

MA-3:
Elev. 226.5 m to 224.5 m

MA-4:
Elev. 229.6 m to 228.8 m,
Elev. 227.7 m to 227.4 m,
Elev. 227.2 m to 226.5 m and
Elev. 225.8 m to 224.3 m

MA-5:
Elev. 227.2 m to 226.1 m

MA-6:
Elev. 228.6 to 228.2 m

0 m 0.25 m 0.50 m

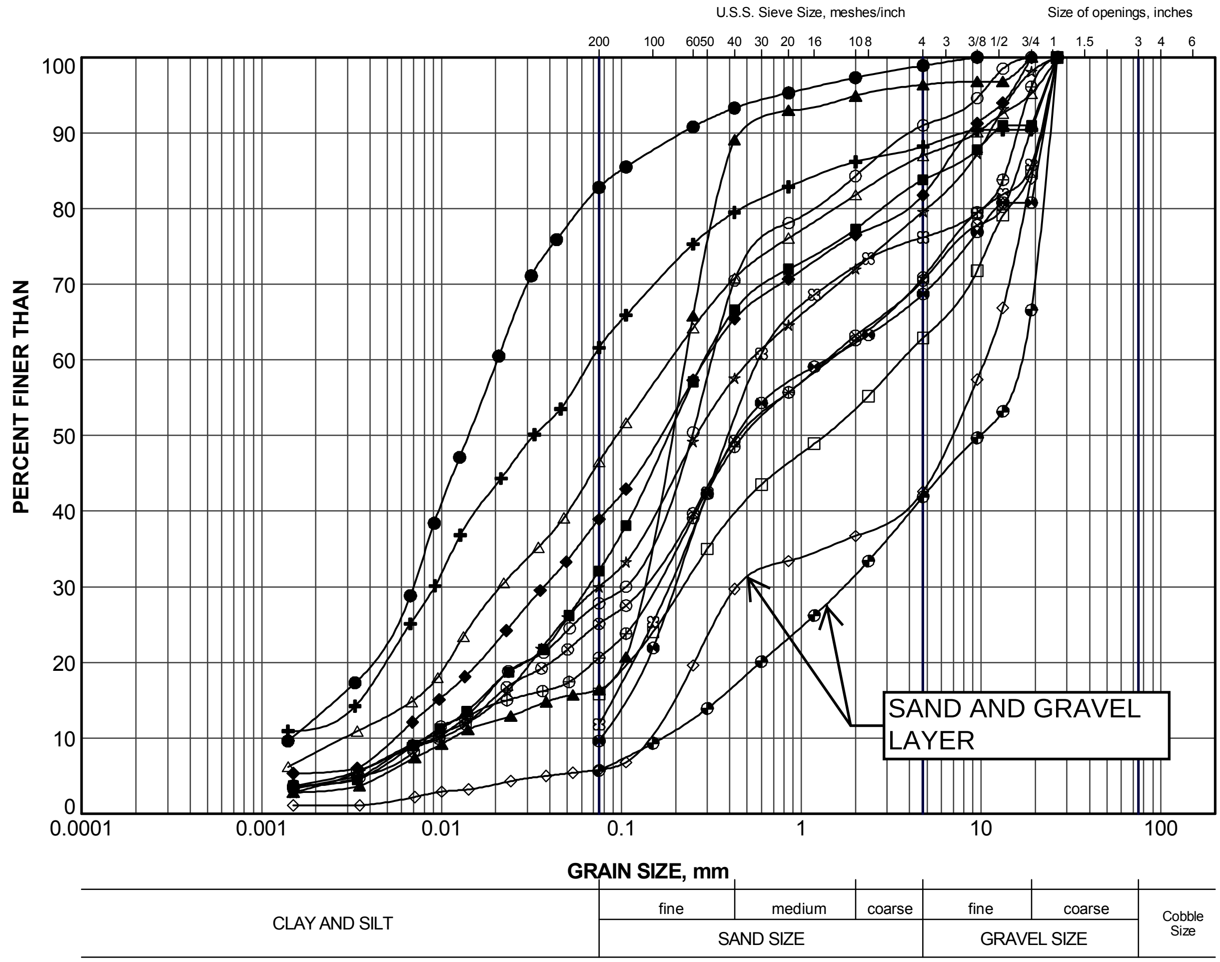
0 ft 1 ft 2 ft

PROJECT
MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11

TITLE
**ROCK CORE PHOTOS**

Golder Associates

| | | | |
|---|---|---|---|
| PROJECT No. | 10-1191-0038 | | FILE No. ---- |
| DESIGN | RH | 11 Nov. | SCALE AS SHOWN REV. |
| CADD | -- | | **Figure C3** |
| CHECK | AB | 11 Nov. | |
| REVIEW | FJH | 11 Nov. | |




## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|---|---|---|---|
| ● | MA-1 | 4 | 230.5 |
| ■ | MA-1 | 7 | 228.1 |
| ▲ | MA-1 | 11 | 222.2 |
| ✚ | MA-2 | 1 | 230.2 |
| ◆ | MA-2 | 3 | 228.7 |
| ◇ | MA-2 | 7 | 227.1 |
| ○ | MA-2 | 9 | 225.6 |
| △ | MA-3 | 1 | 230.2 |
| ⊗ | MA-3 | 3 | 228.6 |
| ⊕ | MA-3 | 5 | 227.1 |
| □ | MA-4 | 4 | 228.5 |
| ◉ | MA-4 | 7 | 227.1 |
| ◕ | MA-5 | 3 | 228.0 |
| ☆ | MA-6 | 7 | 229.5 |
| ⌘ | MA-6 | 10 | 226.5 |

PROJECT

MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11

TITLE

# GRAIN SIZE DISTRIBUTION

SAND AND SILT (TILL)

| | | | | |
|---|---|---|---|---|
| PROJECT No. | | 10-1191-0038 | FILE No. 1011910038 BH LOGS.GPJ | |
| DRAWN | JJL | Nov 2011 | SCALE N/A | REV. |
| CHECK | AB | Nov 2011 | FIGURE C4 | |
| APPR | FJH | Nov 2011 | | |

Golder Associates
SUDBURY, ONTARIO

LDN_MTO_GSD-15 GLDR_LDN.GDT

# APPENDIX D

## Non-Standard Special Provisions and Operational Constraints

**<u>CSP FOR INTEGRAL ABUTMENTS – Item No.</u>**

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Non-Standard Special Provision

---

***Scope***

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

**SUBMISSION AND DESIGN REQUIREMENTS**

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

**MATERIAL**

***Corrugated Steel Pipe***

CSP shall be in accordance with OPSS 1801 and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract Drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract Drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

***Sand Fill***

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

**Table 1 – Sand Fill Gradation Requirements**

| **MTO Sieve Designation** | | **Percentage Passing by Weight** |
|---|---|---|
| 2 mm | #10 | 100% |
| 600 µm | #30 | 80% to 100% |
| 425 µm | #40 | 40% to 80% |
| 250 µm | #60 | 5% to 25% |
| 150 µm | #100 | 0% to 6% |

**CONSTRUCTION**

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to the design tip elevation or bedrock if end-bearing piles are selected.
4. Place loose sand into the CSP.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the top of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

| **Criteria** | **Tolerance** |
|---|---|
| **Maximum deviation of CSP from pile centroid** | **+/- 50 mm** |
| **Maximum deviation of any point on the top perimeter of the CSP from the specified elevation** | **+/- 10 mm** |

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

***Basis of Payment***

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

**<u>SUBGRADE PROTECTION - Item No.</u>**

---

Non-Standard Special Provision

---

Where pile caps or spread footings are constructed, the sand and silt till subgrade will be susceptible to softening and degradation on exposure to water and construction traffic. If the concrete for the foundation cannot be poured within four hours after inspection and approval of the subgrade, a working mat of lean concrete or mass concrete, with a minimum thickness of 100 mm, should be placed on the foundation subgrade.

Lean concrete shall have a compressive strength of at least 5 MPa, and be placed in accordance with OPSS 904.

**Basis of Payment**

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

## GROUNDWATER CONTROL - Item No.

---

Non-Standard Special Provision

---

Foundations for the piers will require excavations to extend below the water level. Cohesionless soils (sand and silt till) that are present below the groundwater table will slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. The Contractor is to design and install an appropriate dewatering system for the foundations to enable construction in dry conditions, and prevent disturbance to the founding soils.

### Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

**OBSTRUCTIONS**

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Operational Constraint

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As part of the work for the installation of foundations, the Contactor shall be alerted to the presence of cobbles and boulders within the sand and silt till.

The Contractor shall be alerted that a dead-man tie-back anchor system supports the existing sheet-pile retaining wall at each end of the bridge.

The existing support system should be maintained and kept stable during the various stages of excavation and foundation installation.

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